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DESIGN STANDARDS FOR STRUCTURAL STEEL DOLPHINS IN COHESIONLESS --ETC(U)
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Design standards for structural steel dolphins in cohensionless soils discuss soil characteristics, dolphin loads, rated capacities and structural behavior for single and multiple-pile dolphins. Examples of design for rated force and energy capacities are given, together with the steel grades and stress levels, pile spacing and embedment lengths.

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DESIGN STANDARDS FOR STRUCTURAL

STEEL DOLPHINS IN COHESIONLESS SOILS

NAVAL FACILITIES ENGINEERING COMMAND

DEPARTMENT OF THE NAVY

January 20, 1974

ABSTRACT

Presented is the following Naval Facilities Engineering Command (NAVFAC) publication, entitled:

"Design Standards for Structural Steel Dolphins in Cohesionless Soils".

The contents include:

1. Foreword
2. Preface with Introductory Remarks
3. The report on Design Standards with subgrade characteristics and rated capacities of piles, and behavior of dolphins. Examples of design for different stress levels, pile spacing and their embedment lengths are given.

This publication was prepared by the NAVFAC Engineering Investigation (EI) program on 20 January 1974.

FOREWORD

"DESIGN STANDARDS FOR STRUCTURAL STEEL DOLPHINS"

Considerable collision damage to ships and structures occurs each year in ports and harbors often because of inherent rigidity of piers, wharves and platforms. A ship depending on its mass, velocity of approach, and its motions caused by winds, currents, tides and waves may represent considerable kinetic energy, destructive upon contact. This energy has to be absorbed by the elastic deformation of the ship's hull and the structure itself, after employed fendering devices take care of their share up to the limit. It would, therefore, be desirable that the absorption of the ship's kinetic energy, which began by camels in some cases and fenders, continues further until fully dissipated, preferably by the structure, without its destruction or serious damage. A flexibility of the structure would thus be a prerequisite, particularly when fendering turns out to be inadequate. Toward that goal "Design Standards for Structural Steel Dolphins" are provided.

INTRODUCTORY REMARKS TO

DESIGN STANDARDS FOR STRUCTURAL STEEL DOLPHINS

1. The report covers single pile and multiple pile-dolphins. The single pile sizes in tables of Fig. 14 and Fig. 13 are 36", 48" and 60", 72" in diameters, respectively, with a wall thickness from 0.75" for all sizes to 1.5" for 60" and 72" diameter piles.

For increased capacities, multiple (small diameter 36" and 48") pile-dolphins comprise two, three, four and five piles as well as six and seven piles of the 48" diameter for different depths of water - providing a diversified range of rated static force, and elastic energy absorption. This is for a case in which the structure could be loaded at any location around its perimeter.

Different combinations of pile sizes and their numbers in addition to those shown in the table of Fig. 14, could be used as desired or warranted by the local conditions.

2. These structures do not include fendering systems. The fenders, if any could be provided in accordance with the specific requirements of using activity. In such a case, the energy absorbing capacity of the fenders should be added to the listed rated elastic energy of the dolphins, shown in the table, Fig. 14, to arrive at the total energy capacity of the structure. The EI project "Fendering for Structural Steel Dolphins" is available but fenders could be individually designed as needed.

3. Dolphin piles do not carry axial stresses (tension or compression) but act basically in bending.

Since the connections of piles in a group (to exclude the axial pile loads) are expensive, it is recommended that a single larger diameter pile be used for a required capacity, rather than a group of smaller size piles, in locations where large size piles and driving equipment are available.

Usually a single-pile-dolphin should suffice, provided it can be obtained in the required size. This would constitute the simplest and least expensive solution. In

special situations, the two-pile-dolphin with simple connecting member, and fendering if required, could provide a suitable and inexpensive structure, as depicted in Fig. 11 and Fig. 12.

4. To assist the designer and the engineer, 13 various examples are given in the text to facilitate use of the report. To name a some:

a. For existing dolphins - calculating the rated capacity and the energy absorption for a given mooring force at a given elevation, rotation, deflection, and forces in bracing.

b. For design of dolphins - determining the required number of available piles with the known properties -

(1) for a given mooring force acting at a given elevation.
(2) for a required energy absorption under the known loading.

To provide more favorable distribution of forces from piles to soils, a concept of a sleeved pile is currently being investigated by NAVFAC for future applications.

Verification of the calculated values for sleeved piles fron this EI project and expected behavior of structural members and soils will be undertaken in full size field testing.

5. The use of the standard designs, the structural steel stress levels, details of connections, pile spacing, their embedment and installation are discussed in the report.

The derivation of equations and references to charts used in the report are provided in Appendix A.

This report covers structures in cohesionless soils, however, consideration is being given to the preparation of similar aids for other soil conditions in future.

Casimir J. Kray
Consultant,
Waterfront Structures

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LIST OF SYMBOLS

Symbol		Definition	Units
A_E	...	A factor, dependent on pile diameter and pile wall thickness, used to determine dolphin rated energy capacity	in^2
C_E	...	A factor, dependent on pile yield stress, and on ratio T_{min}/H, used to determine dolphin rated energy capacity	$kips/in^2$
C_M	...	Pile moment coefficient	
C_Δ	...	Coefficient in expression for maximum pile deflection	
C_θ	...	Coefficient in expression for maximum pile slope change	
D	...	Pile (outside) diameter	ft
E	...	Modulus of elasticity	$kips/ft^2$
F_{ch}	...	Design force in chain	kips
F_{st}	...	Design force in inter-pile struts	kips
F_{max}	...	Maximum force on dolphin, for rated energy capacity	kips
F_R	...	Dolphin rated force capacity	kips
H	...	Height of load point, above seabed	ft
I	...	Moment of inertia of pile section	ft^4
K_p	...	Passive pressure coefficient	
M	...	Pile maximum bending moment	kip-ft
M_y	...	M causing yield stress in pile outer fibers	kip-ft
N	...	Number of piles in a dolphin cluster	
N_c	...	Number of chain-connected piles	
Q	...	Eccentricity moment, about dolphin axis	kip-ft
S	...	Pile section modulus	in^3
T	...	Pile characteristic length (= relative stiffness factor in NAVFAC DM-7, 1971)	ft
T_{min}	...	T, based on maximum soil stiffness	ft
T_{max}	...	T, based on minimum soil stiffness	ft
W_R	...	Rated energy capacity of dolphin	kip-ft
e	...	Eccentricity of applied dolphin load, with respect to dolphin axis	ft
f	...	Coefficient of horizontal subgrade reaction	$kips/ft^3$; lbs/in^3
f_y	...	Yield stress	$kips/in^2$
r	...	Radial distance from dolphin axis to axis of a perimeter pile	ft
s	...	Spacing, c.c., of piles in a chain-connected pair	ft

LIST OF SYMBOLS (Cont.)

		Units
t	... Nominal thickness of pile wall	in
t_n	... Net pile wall thickness with 0.125" deduction for corrosion	in
α	... Horizontal angle defining direction of applied dolphin load	degrees
Δ	... Horizontal deflection of pile at top	ft
Δ_{max}	... Maximum deflection of pile at top	ft
γ_s	... Density of submerged soil	lbs/ft^3
θ	... Pile-top slope change	radians
θ_{max}	... Maximum pile top slope change	radians
Φ	... Soil friction angle, based on effective stress	degrees

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LIST OF FIGURES AND TABLES

DESIGN STANDARDS FOR STRUCTURAL STEEL DOLPHINS

1.0 INTRODUCTION

These standards provide dolphin designs for six different capacities (mooring force and energy absorption) in each of four water depths, for seabeds of cohesionless soils. In addition methods are presented for the design of dolphins of other capacities, other water depths, using other pile sizes and, or, other steel strength grades. These methods likewise are limited in their applicability to dolphins embedded in seabeds of cohesionless soil. Tables and graphs, and numerical design examples illustrating their use, are presented. The examples, to assist the field engineer, include determinations of dolphin force and energy capacities, pile-top deflection and slope, and forces in inter-pile connecting struts and chains.

2.0 SCOPE

The specific dolphin designs herein presented incorporate pipe piles of only two sizes (36 PP.75 and 48 PP.75), in one strength grade (60 ksi yield), in clusters of 1 to 7 piles. These designs provide the following ranges of mooring force capacities and energy capacities.

Water Depth	Mooring Force Capacities	Energy Capacities
40 ft.	27 to 191 kips	30 to 179 ft-kips
50 ft.	23 to 204 kips	32 to 240 ft-kips
60 ft.	36 to 214 kips	52 to 310 ft-kips
70 ft.	32 to 221 kips	55 to 386 ft-kips

The design of fendering was not within the scope of the contract for the development of these standards. Energy capacities referred to herein are the elastic energies absorbed by the steel piles (and, to a lesser extent, the soil). In most cases the required energy capacities are thus developed at force magnitudes which can be distributed over the ship's hull by simple timber fendering. When special energy-absorbing fendering is used, the total energy capacity is the sum of the fender energy capacity plus the elastic energy capacity as given herein.

The dolphins covered by these standards are comprised of one or more steel pipe piles functioning primarily as vertical beams cantilevered from the seabed. When more than one pile is used their interconnection for effective group action can be costly. For this reason, a single large pile is more attractive whenever piles of the required size, and pile-driving equipment of the required capacity, are readily available. In accord with instructions from the Naval Facilities Engineering Command, the pile diameters in the standard dolphins presented herein are limited to 4 feet. However, the capacities of single-pile dolphins of 5-foot and 6-foot diameters are are tabulated in Fig. 13. Further, methods of design presented herein, and charts to facilitate their application, are suitable for a wide range of pile sizes, steel strength grades, water depths, and dolphin capacities.

The standard dolphins and the design methods herein all assume a seabed of cohesionless soil.

3.0 SOIL CHARACTERISTICS

3.1 SOIL TYPE

These standards are applicable to dolphins at sites where the seabed is comprised of essentially uniform cohesionless soil throughout the pile embedment length. For design purposes the soil characteristics of interest are the coefficient of variation of soil modulus of horizontal subgrade reaction with depth, f, the coefficient of passive soil pressure, k_p, and the submerged unit weight of soil, γ_s.

3.2 NUMERICAL VALUES OF SOIL CHARACTERISTICS

Fig. 1 provides values of f and k_p as a function of sand density. Values of f shown on Fig. 1 were obtained by reducing the values of f for coarse grained soils from Fig. 11-8 of NAVFAC DM-7 by 60% (approx. ratio of γ_s to γ_{dry}) to account for the effects of submergence in accordance with recommendations presented by Terzaghi.(1) The detailed dolphin designs presented in these standards are based upon a medium sand, for which f varies from 8 to 24 lbs/inch^3 and k_p varies from 3.0 to 4.0.

Where the use of a low value of f is conservative, the mean value, reduced by 75 percent for the effect of repeated loading, has been used; i.e., $f_{min} = 0.25(16) = 4.0$ lb/inch^3; where the use of a high value of f is conservative, the maximum value for the range, unreduced for repeated loading, has been used; i.e., $f_{max} = 24$ lbs/inch^3. It always is conservative to use a low value of k_p, and the lowest value for the medium sand range, $k_p = 3.0$, has been used in these standards. The submerged soil density, γ_s, varies only over a very narrow range, and the conservatively low value, $\gamma_s = 50$ lb/ft^3, has been used in these standards.

Where the seabed is comprised of cohesionless soil, but of substantially different density than herein assumed, the range of values of f and k_p should be determined. For dolphin design f_{min} should be taken equal to 25 percent of the lowest value at the site, and f_{max} should be taken equal to the highest value at the site. The minimum value of k_p for the site should be used in design.

3.3 SUBSURFACE INVESTIGATION

In some instances the subsurface soil characteristics will be known from prior investigations. When such data is not available, the following investigations are necessary.

At the location of each dolphin, at least one 2-1/2" dia. test boring should be conducted. Standard split-spoon samples (1-3/8" I.D. and 2" O.D.) should be obtained from the mudline to the top of the sand stratum. For dolphin design purposes, H, "the height of load above seabed," must be taken as the height of load above the top of the sand stratum, and the determination of the latter elevation is necessary. Within the sand stratum, samples should be obtained at 5-ft. intervals to a depth of 10 ft. below the bottom elevation of the proposed piles.

Standard penetration tests should be conducted according to the procedures specified in Table 4-2 of NAVFAC DM-7, which specifies that the split-spoon sampler should be driven with a 140-pound hammer falling freely through a distance of 30 inches, recording the number of blows for

each six inches of penetration while driving the spoon a total distance of 18 inches, unless indicated otherwise on Table 4-2. The total number of blows required to drive the spoon through its final 12 inches of penetration is referred to as the blow count, N.

For cohesionless soils, the required soil parameters are a function of relative density, which is related to the blow count, N, as shown on Fig. 4-2 of NAVFAC DM-7. $\overline{\phi}$ for cohesionless soils (SW + SP) can be found as a function of relative density from Fig. 3-7 of NAVFAC DM-7, and $k_p = (1+\sin\overline{\phi})/(1-\sin\overline{\phi})$. With k_p determined, the value of f can be determined from Fig. 1, herein.

4.0 SOURCES OF DOLPHIN LOADS

Primary (horizontal) loads on a dolphin can arise from its use as a ship mooring, from its function to absorb kinetic energy of a ship during docking operations, or from its use in assisting the maneuvering of a ship in a narrow channel. In addition to these primary loadings the dolphin may be subjected to significant forces associated with waves and water currents. The dolphin may be used for mooring purposes under storm conditions, but it is unlikely to be used for docking operations, or to assist in ship maneuvers, under severe storm conditions. The mooring force for which the dolphin is to be designed must reflect maximum wind, wave, and water current forces on the ship as well as wave and water current forces on the dolphin itself. Wave and water current forces on the dolphin may be converted to an equivalent force (i.e., the force at water surface which would cause the same overturning moment about a point on the seabed), and added directly to the force applied by mooring line or by ship contact.

Taking account of all of the above load sources, the dolphin design is entered with (a) a maximum force to be resisted, (b) a maximum energy to be absorbed, (c) a maximum contact force which can be tolerated without damage to the ship's hull. The latter force limit may be a function of the fender area that can be contacted by the hull. Since the dolphin, typically, will be intended to serve a range of vessels with different characteristics, there may be several loading conditions to be satisfied.

5.0 RATED CAPACITIES AND FACTORS OF SAFETY

The principal structural elements are pipe piles of ductile steel utilized primarily in the bending mode. The bending moment corresponding to steel yield can be computed with considerable confidence, and pile embedment lengths are more than adequate to preclude "failure" prior to steel yield. Accordingly the pile top lateral force required to cause yielding of the pile steel can be reliably determined. It should be noted that the time-dependent reduction in this force, due to loss of material through corrosion, is accounted for by a 0.125" reduction in the pipe wall thickness used in strength computations. The method of connecting the several piles in a dolphin cluster is such that all share, essentially equally, in resisting the load.

Rated Force Capacity of the dolphin is defined as 50 percent of the dolphin force which, when equally distributed among the piles, will develop yielding of the pile steel. Since the bending moment required to cause actual pile failure is substantially greater than the bending moment to initiate yielding, the rated force capacity incorporates a factor of safety somewhat greater than 2.0.

Energy absorption is a function of both load and deflection. Assuming maximum soil stiffness (f_{max}), the pile lateral load and deflection corresponding to a maximum bending stress equal to 75 percent of the yield stress are determined. The Rated Energy Capacity is defined as one-half the product of load and deflection, thus computed. If the soil were linearly elastic, and of the assumed maximum stiffness, up to pile yield, the rated energy capacity would represent 56 percent (i.e., $(.75)^2$) of the energy absorbed by the pile at the point of yielding. Thus the rated energy capacity would imply a factor of safety of 1.78. However, the actual soil stiffness must be less than the assumed maximum stiffness, particularly under repeated loadings, and the stiffness must decrease with increasing load. For both of these reasons the actual energy absorbed in loading a pile to the yield stress will exceed twice the rated energy capacity; that is, the factor of safety with respect to energy is somewhat greater than 2.0. Of course in an accident condition of high

overload the pile steel yields, and very much larger amounts of energy will be absorbed in the plastic range. The inherent ductility of steel piles thus provides assurance that accidentally high levels of vessel kinetic energy can be absorbed, though these may result in dolphin damage that necessitates replacement.

6.0 STRUCTURAL BEHAVIOR OF DOLPHINS

6.1 SINGLE-PILE DOLPHINS

Within its service load range the single-pile dolphin is treated as a vertical cantilever beam supported on an elastic foundation. The foundation modulus is assumed to increase linearly with depth. Thus the local soil reaction (lbs. per foot of pile length) at any depth is the product of the pile lateral deflection at that depth, the depth, and f, the coefficient of variation of the soil modulus with depth.

It should be noted that, for any given lateral load at the top of the pile, the magnitude of the pile maximum bending moment is very insensitive to the assumed value of the soil coefficient, f. This is due to the fact that, for the water depths of interest ($\geq$ 40 feet), the water depth dominates the effective "moment arm" of the applied load. That is,the maximum bending moment occurs at depth into the soil which is very small compared to the water depth. Accordingly, small changes in this depth into the soil, which accompany changes in the coefficient f, cause only small changes in the bending moment. In this regard the behavior of a dolphin pile is fundamentally different from the behavior of a pile laterally loaded at, or near, this ground surface.

The horizontal deflection of a dolphin pile, at the point where it is loaded, results, in part, from pile flexure above the seabed and, in part, from the deflection and slope at the seabed. The latter contributions are significantly influenced by the magnitude of the soil coefficient, f. Since absorbed energy is proportionate to deflection, the rated energy capacity is conservatively based upon a high value of f (leading to a conservatively small value of deflection).

"Failure" of the pile involves development of a "plastic hinge," and the mobilization of soil passive pressure on the portion of the pile above this hinge. The passive soil reaction (lbs. per foot of pile length) is assumed to be three times the product of soil depth, pile diameter, submerged unit soil weight, and the coefficient of passive soil pressure. Because of the substantial distance of the point of loading above the seabed, the magnitude of applied load corresponding to an assumed pile hinge moment is very insensitive to assumed values of submerged unit soil weight and coefficient of passive soil pressure. In this regard the dolphin pile differs from piles subjected to lateral load at, or near, the ground surfaces. In the preparation of these standards, γ_s = 50 lbs/ft^3 and k_p = 3.0 have been assumed. These are conservatively small values, but substantially different values would have changed the results by only a few percent.

For the ductile steels appropriate for dolphin piles, the bending moment at the plastic hinge can be expected to exceed the bending moment associated with the initiation of yielding. However, the yield moment has been, conservatively, taken as the hinge moment. The rated (mooring) force capacity of the dolphin pile, assumed applied 10 feet above the water surface, is defined as 50 percent of the load required to develop the yield bending moment.

6.2 MULTIPLE-PILE DOLPHINS

The single-pile dolphin is the best choice when a pile of the required size, and equipment with the capacity to drive it, are readily available. However, there will be cases in which two or more steel piles must be combined to form a dolphin of the required capacity. In contrast to timber dolphins, which often are comprised of a very large number of timber piles, steel dolphins rarely will require more than a few piles. Because the piles function primarily in bending (rather than as tension-compression elements), and to minimize the possibility of damaging contact with the lower part of a ship's hull, the steel piles of a multiple-pile dolphin are arranged in parallel (i.e., all are vertical).

The purpose of inter-pile connections is to distribute the applied load as equally as possible among all the piles, without inhibiting the pile-top slope changes associated with pile bending. Underlying these two objectives are the following considerations:

a) The dolphin capacity is limited by the bending stress in that pile which is subjected to the largest shear and bending moment. The total dolphin force required to produce the allowable bending stress in the most heavily loaded pile is maximized if bending stresses in the other piles concurrently reach, or closely approach, the allowable values.

b) If inter-pile connections inhibit pile-top slope changes, very large bending moments are developed in the piles, and in the framing elements which connect the piles. Because of the short span of the framing (i.e., close spacing of the piles), these bending moments are accompanied by very large, vertical, shear forces in the framing. The framing shear forces are reflected in large axial tension forces in some piles. It is not possible to preclude partial pull-out of such piles from the seabed.

The complexity of inter-pile connections required to achieve the above described objectives is dependent upon the number of piles in the dolphin, how large a portion of the dolphin perimeter can be subjected to loading, and the magnitude of possible eccentricity of the line of action of the load with respect to the dolphin axis. The contract under which these standards were prepared specified dolphins suitable for loading at any point on their perimeter. This requirement, together with the range of load directions implied by friction components at the ship-to-dolphin contact point, creates maximum demands on the system of inter-pile connections. The requirement that loading be accepted at any point on the perimeter lead to axi-symmetric arrangements for all of the multiple-pile dolphin designs herein presented. The necessity to provide for large eccentricity of loading, with respect to the dolphin axis, required special connection elements (i.e., chain-linked torque arms) to mobilize the

torsional resistance of individual piles. Absorption of most of the dolphin torque through torsional moments in the piles minimizes twisting of the entire pile group, with two important consequences. First, and most important, bending stresses are essentially equal in all of the piles, even when the dolphin is eccentrically loaded. Thus there is negligible degradation of the dolphin capacity under eccentric loading. Second, the essentially parallel and equal displacements of the pile tops facilitates the connection of inter-pile struts, which serve to maintain the distances between adjacent piles. These struts can be connected by pins oriented (horizontally) to accommodate pile-top slope changes. If twisting of the pile group were not inhibited, the connection detail would be required to accommodate not only the vertical rotations associated with pile-top slope changes, but horizontal rotations as well.

Figs. 9 and 10 illustrate, for a 3-pile dolphin, the increase in dolphin capacity that can be achieved when the torsional resistance of individual piles is mobilized. In each case the dolphin load F is assumed to act with an eccentricity e with respect to the dolphin axis. Thus the dolphin is subjected to a concentric load F plus a torque eF, as shown in Figs. 9(b) and 10(b). If the dolphin torque must be resisted by pile bending, Figs. 9(c) and 9(d) show the individual pile loads due, respectively, to the dolphin concentric force F and the dolphin torque. Note that pile 1 is far more heavily loaded than either of the other two piles.

If the dolphin torque is resisted entirely by torsion in two of the three piles, as shown in Fig.10(c), the three piles are loaded equally in bending. A comparison of these two cases shows that the capacity of a dolphin which fully mobilizes the torsional resistances of individual piles is $(1 + e/r)$ times the capacity of a dolphin in which dolphin torque is resisted entirely by pile bending. In the factor $(1 + e/r)$ the term r is the distance from dolphin axis to pile axis. The following conclusions should be noted:

a) Potential improvement in the dolphin capacity is directly proportionate to the eccentricity of dolphin loading.

b) If the maximum possible eccentricity is small it may be unnecessary, or economically unattractive, to provide inter-pile connections designed to mobilize the torsional resistances of individual piles.

c) The maximum effective eccentricity of dolphin loading that need be considered is $e = r$. For larger eccentricities the additional torque, $(Fe - Fr)$, is transmitted directly to individual piles by the fendering. Thus the capacity augmentation factor $(1 + e/r)$ reaches a maximum value of 2.0. Clearly it is advantageous to mobilize individual pile torsional resistances when large eccentricities of dolphin loading must be accepted.

The dolphin capacity augmentation factor, $(1 + e/r)$, which has been demonstrated for the 3-pile dolphin, is equally applicable to all of the dolphin designs presented herein except the 7-pile dolphin. For the latter the factor can be shown to be $(1 + \frac{7}{6}\frac{e}{r})$. Thus, for practical purposes, it can be concluded for all multiple-pile dolphins the capacity under large eccentricities of loading theoretically can be approximately doubled by mobilizing the torsional resistance of individual piles. It should be noted that the actual augmentation in capacity will be slightly less than the theoretical value since 100 percent of the dolphin torque will not be resisted by individual pile torsion. The actual percentage resisted depends upon the stiffness of this mode in comparison with the stiffness of the mode which resists dolphin torque through pile bending (Fig. 9). Fortunately the former mode typically is at least an order of magnitude stiffer than the latter. Accordingly when inter-pile connections are designed to mobilize individual pile torsional resistances, it is reasonable to rate the dolphin capacity on the basis of concentric loading. In contrast, dolphins whose inter-pile connections do not include this feature must be assigned a substantially lower capacity rating under eccentric loading than under concentric loading. For cases of large eccentricity the

capacities of such dolphins will be only 50 percent of the concentric load ratings.

6.3 MULTI-LEVEL DOLPHIN LOADING

At many sites the tidal range will be small enough to permit fender attachment at a single elevation on the dolphin. At other sites the tidal range may require fender attachment at two (or, more rarely, at three) different elevations. If the dolphin is comprised of a single pile, the provision of fender attachments at more than one elevation presents no particular design problems. The designer should note, however, that the rated energy capacity decreases somewhat with decreases in elevation of dolphin loading above the seabed. For the water depths of interest the rated energy capacities, at lowest and highest probable fender attachment elevations, rarely will differ by more than 15 percent; in most cases the difference will be much less.

In a multiple-pile dolphin the existence of fender brackets at more than one elevation correspondingly may require more than one system of inter-pile connections. Typically the major system (including components for mobilizing individual pile torsional resistances to dolphin torque associated with eccentric loading) will be located at the upper bracket elevation. If no lower level system is provided, one of the piles may be subjected to the entire dolphin capacity load, at a lower bracket, with assistance from the other piles occurring only at the upper end of the loaded pile. Under this condition the loaded pile may experience substantially greater bending stress than the other piles. This maldistribution of stresses among the piles would require a reduction in the dolphin rated energy capacity to avoid overstress of the loaded pile. The extent to which the loaded pile can experience significantly higher bending stresses than the other piles will increase (a) with the total number of piles in the dolphin, and (b) with the ratio of the distance between upper and lower fender brackets to the water depth.

In some instances (small number of piles and/or small ratio of fender bracket spacing to water depth) the reduction in dolphin rated capacity associated with loading on the lower bracket may be acceptably small

(e.g., 10 percent, or less). Moreover, contact by the larger vessels may be either impossible, or unlikely, under the low tide conditions which lead to loading on the lower fender brackets. In those cases where reduction in dolphin rating, due to overstress in the loaded pile, is not acceptable, an additional system of inter-pile connections should be provided at the elevation of the lowest set of fender brackets. In some cases this system need only be comprised of a system of inter-pile struts which maintain the c. to c. spacing of piles at that elevation. In a few cases it also may be necessary to incorporate elements which mobilize individual pile torsional resistances. An example of the latter is a 2-pile dolphin subjected to loading in a direction essentially normal to the vertical plane common to the two pile axes. Such a loading applied to one of the two piles, at a lower bracket, may overstress the contacted pile. To avoid such overstress by lower level inter-pile connection, this connection must include elements mobilizing individual pile torsional resistances.

7.0 DOLPHIN DESIGN

7.1 NUMBER AND ARRANGEMENT OF PILES

All of the standard dolphin designs presented herein are comprised of piles in axi-symmetric arrangements. In each of these standard designs the piles are either 36PP.75 or 48PP.75. The arrangements reflect the specified requirement that these standard dolphins be able to accept loading at any point on their perimeters. Two pile sizes were selected in the interest of standardization.

When a particular dolphin installation requires equal capability at all points on the perimeter, a selection from the standard designs may be appropriate. Alternatively a single-pile dolphin, using a large size pile, may be preferred. Fig.13 presents a tabulation of capacities for large size single-pile dolphins .

In many cases the dolphin will not be required to accept loading at all points on its perimeter. Indeed the primary purpose may be to assist

in docking, to assist a ship in its passage through a narrow channel, or to protect a facility from contact by a ship. In each of these common cases only a very limited portion of the dolphin perimeter is subject to loading, and the range of directions of possible loading is very narrow. Numbers and arrangements of piles suitable for these cases may differ from those presented in the standard dolphin designs.

7.1.1 SINGLE-PILE vs. MULTIPLE-PILE DOLPHINS

Whenever a single-pile dolphin can be used, it should be seriously considered. Its particular attraction lies in the fact that it eliminates both the initial costs and the maintenance costs associated with inter-pile connections. These are major advantages. However, it should be noted that, for equal rated energy capacities, a large single-pile dolphin will develop a larger contact force than will a multiple pile dolphin comprised of smaller piles. The larger contact force may introduce additional costs in the details of fendering required to maintain contact pressure on the ship's hull within acceptable values. Fender details are not within the scope of the contract under which these standards were prepared. However, it may be noted that a large contact force may only require a correspondingly large fender contact area. Where the provision of a sufficiently large contact area is not practical, it may be necessary to absorb a portion of the energy in the fenders themselves, thereby reducing the required dolphin energy rating and the associated contact force.

In some cases it may be desired to provide constraints to ships' lateral motion at several points along a channel (e.g., at a bend in the channel), as shown in Fig.11 . It may be desirable to provide a single pile at each point, but to interconnect two or more successive piles to increase the energy capacity available at contact with any individual pile. Thus a multiple-pile dolphin design would derive from the aim of providing maximum rated energy at spaced points. In Fig. 11(b) fenders are provided at each pile, but force applied to one pile is shared by the pile and by one of its neighboring piles. In Fig. 11(c) a simply-suppor-

ted fender beam (or truss) is mounted on each successive pair of piles, and fenders are attached at mid-length of each fender beam. Thus loading on any fender automatically is transmitted to two piles.

In the arrangement of Fig. 11(b) the major function of inter-pile connecting elements is to transmit horizontal shear, and concurrent mobilization of the torsional resistances of individual piles is fundamental to the concept. In contrast, the arrangement of Fig. 11(c) does not require mobilization of individual pile torsional resistances. Pile torsional strength is not governed by torsional shear stress in the pile steel, but by pile-to-soil friction limits. These limits will restrict the spacings of successive piles for which the arrangement of Fig. 11 (b) can be used. There is no corresponding limit on the arrangement of Fig. 11(c).

7.1.2 PILE ARRANGEMENTS

Fig. 12 illustrates possible pile arrangements when a series of individual dolphin contact points are required, and the applied loading is narrowly limited in direction. Where the purpose is to protect a facility, or to assist in docking, the contact points often lie on a straight line parallel to the facility. In these cases, the horizontal angle between the line of dolphin contact points and the side of the vessel typically will be small. Thus the component of loading perpendicular to the line of contact points may be very much larger than the component parallel to this line. The latter component will result mainly from rubbing friction between ship's hull and fenders. Fig. 12 illustrates pile arrangements which should be considered for this case.

Fig. 12 (a) illustrates a simple arrangement of single-pile dolphins, grouped, where convenient, along the contact line. Fig.12 (b) shows an arrangement of 2-pile dolphins, with compression strut interconnections between front and rear piles. End connections of strut to piles must be detailed to provide articulation in both the horizontal and the vertical planes. The (small) component of loading parallel to the protected facility will be resisted by the foreward piles only, and will displace these piles

(relative to their rear pile neighbors) parallel to the contact line. This relative deflection must be accommodated by the end connections of the struts connecting front and rear piles. Note that this relative displacement of front and rear piles can be essentially suppressed, if desired, by interconnecting them not only with struts but also with elements which mobilize their individual torsional resistances.

If the space between vessel and protected facility is too narrow to accommodate the front-and-rear-pile dolphins illustrated in Fig.12 (b), an arrangement of 2-pile or 3-pile dolphins with the piles parallel with the protected facility may be indicated. Such a (3-pile) dolphin layout is shown in Fig. 12(c). Here the inter-pile connections must include elements to mobilize individual pile torsional resistances, since the aim is to mobilize the capacity of additional piles when the fendering on any pile is contacted. Although the (small) load component parallel to the protected facility can be accommodated by a single pile, tension-compression struts must be included in the inter-pile connections. The purpose of these struts is to prevent changes in c. to c. pile spacings, which cannot be accommodated by the elements designed to mobilize pile torsional resistances.

The arrangement shown in Fig. 12 (d) is similar to that shown in Fig. 11 (c). Here the fenders are mounted at midlength of a fender beam, which is supported on a pair of piles. Note that a horizontal truss may be substituted for the fender beam. Note also that the beam (or truss) need not lie entirely outside the front face of the piles, as shown (schematically, only) in Fig. 12 (d). In application the beam (or truss) would be framed in the space between the adjacent piles. End connections of the fender beam (or truss) to the piles must be detailed to accommodate the pile slope changes associated with the (small) loading component parallel to the protected facility. These connection details present no difficulty because the deflections perpendicular to the facility are essentially identical for the two piles.

The applications presented in this and the preceding section obviously do not exhaust the pile combinations and configurations which may

merit consideration in specific situations. Starting with the site and operational conditions (dolphin purpose, vessel approach angles, vessel sizes and velocities, stream current velocities, wind velocity wave and tidal extremes, waterway geometry, and any other geometrical constraints) the engineer determines

a) magnitude of mooring forces, if any, to be resisted,

b) magnitude of vessel kinetic energy to be absorbed (as elastic strain energy) by the dolphin,

c) geometrical limitations, if any, on dolphin size and shape.

Having the above basic input information, the engineer can use charts contained herein for rapid determination of several alternative combinations of numbers and sizes of piles which will form a dolphin of the required capacity, independent of the shape of the dolphin which will be subsequently chosen. If none of the standard dolphin designs is appropriate for the specific site and dolphin purpose, he should next match the alternative combinations selected on the basis of capacity to the site geometry constraints: i.e., establish tentative pile group configurations. As hereafter described, he should next determine forces to be resisted by the inter-pile connections. Preliminary design of these elements sufficient to serve as the basis of cost comparisons among alternatives should follow. The final choice may reflect the proximity of pile fabricators, relative availability of alternate pile sizes in desired steel grades, and the effect of pile size on required construction equipment capacities.

7.2 DISTRIBUTION OF DOLPHIN FORCES AMONG PILES

The charts herein presented, as design aids, are applicable to a wide range of assumptions regarding the pile group configurations. It will be noted that the charts permit the determination of rated capacities of either a single pile or a total group of N piles. In some cases N appears within specific equations. In either event the capacity of a single pile obviously can be found by taking N = 1.

Where N represents the number of piles in a group, it is assumed that the mooring force (or elastic energy) is shared equally by all N piles in the group under consideration. This implies that the load will, in fact, be applied equally to the piles (e.g., by a fender beam as shown, schematically, in Fig. 12(d), for example), or that the piles will be inter-connected to force essentially equal sharing of the load, regardless of the location of the point of application and the direction of the load on the dolphin. As was described in Section 6.2, the latter can be accomplished by including in the inter-pile connections elements which mobilize individual pile torsional resistances to twisting of the dolphin about its vertical axis. Thus an early design decision involves a choice among three approaches to minimization of possible inequalities among the pile bending moments. The choices are

a) To use unconnected individual piles. (Suitable only when the desired vessel contact points can be so located that each pile's share of the total demand is known. For some layouts it may be clear that only one pile can function at a time, as in Fig. 11(a). In other cases the vessel approach angle, relative to the line of piles, may be so small as to preclude substantial inequalities of the pile deflections, and loads.)

b) To use multiple-pile dolphins with fenders so located as to preclude any significant eccentricity of loading. (This frequently will be feasible when the vessel contact point can be located at a single point on the dolphin perimeter, and the direction of the dolphin load falls within a narrow angular range. In some cases it may be necessary, or desirable, to achieve this result by mounting the fender on a horizontal beam which is simply-supported on spaced individual piles or pile sub-groups, as illustrated, schematically, in Figs. 11(c) and 12(d).)

c) To use multiple-pile dolphins with piles inter-connected in a manner to force equal sharing of the dolphin load. (This is the only solution if loading must be accepted around a major portion of the perimeter. It may be a desirable solution whenever the load can occur at two or more different points on the dolphin.)

In terms of simplicity of fabrication and construction, approach (a) is preferable to (b), and (b) is preferable to (c). However, there will be circumstances (e.g., limitations on availability of piles of large size and/or requirements of location, magnitude, and direction of loading) which dictate the choice of (b) over (a) or (c) over either (a) or (b).

7.3 DESIGN FOR RATED FORCE CAPACITY

As was described in Section 6.1, the rated (mooring) force capacity of a pile, F_R, is defined as 50 percent of the force associated with a plastic hinge within the embedded portion of the pile and development of passive soil pressure above the hinge. The hinge moment is conservatively defined as the bending moment, M_y, which would produce an extreme fiber stress equal to the yield stress, with the pile nominal wall thickness reduced 0.125" to allow for corrosion. The input variables (known or assumed) are

- D, nominal outside diameter of pile, feet
- t, nominal pile wall thickness, inches
- f_y, steel yield stress, ksi
- H, height of loading above seabed, feet

The computed quantities are

- t_n, pile wall thickness reduced for corrosion, inches
- $\frac{M_y}{D}$, pile yield bending moment (kip-feet) divided by pile nominal diameter (feet).

The equations for t_n and M_y/D are

$$t_n = t - 0.125 \tag{1}$$

$$\frac{M_y}{D} = 3\pi f_y \, t_n \left(D - \frac{t_n}{4}\right) \tag{2}$$

Entering Fig. 2 with M_y/D, the value of F_R/ND is read from the curve corresponding to the H of interest. Then, for the N of interest (for a single pile, N = 1),

$$F_R = ND\left(\frac{F_R}{ND}\right) \tag{3}$$

The derivation of Eq. (2) and the basis of the curves of Fig. 2 are presented in an appendix.

EXAMPLE 1

Given a cluster of 3 piles, each pile having a diameter of 3 feet, a thickness of 1.25 inches, and a yield stress of 42 kips/inch^2, what is the dolphin rated capacity for a mooring force applied 60 feet above the seabed?

$$t_n = t - 0.125 = t_n = 1.125''$$

$$\frac{M_y}{D} = 3\pi(42)(1.125)(3 - 0.28) = 1212 \text{ kip-feet/foot}$$

From Fig. 2, for M_y/D = 1212 kip-feet/foot and H = 60 feet, read

$$\frac{F_R}{ND} = 9.4 \text{ kips/foot}$$

$$\therefore \quad F_R = ND = 9.4(3)(3) = \underline{F_R = 85 \text{ kips}}$$

EXAMPLE 2

Given that a dolphin must provide a rated mooring force capacity of 220 kips applied 80 feet above the seabed. How many piles are required if each pile is 5 feet in diameter, has a thickness of 2 inches, and has a yield stress of 50 kips/inch^2?

$$t_n = t - .125 = t_n = 1.875''$$

$$\frac{M_y}{D} = 3\pi(50)(1.875)(5 - 0.47) = 4004 \text{ kip-feet/foot}$$

From Fig. 2, for M_y/D = 4004 and H = 80, read

$$\frac{F_R}{ND} = 22.6 \text{ kips/foot}$$

$$\therefore \quad \frac{F_R}{N} = 22.6\ D = 22.6(5) = 113 \text{ kips}$$

$$\therefore \quad N = \frac{220}{113} = N = 1.95; \text{ use } \underline{N = 2 \text{ piles}}$$

Values of yield stress in excess of 60 kips/inch^2 are not recommended. Values of gross wall thickness, t, less than 0.375", or less than 1/60 of the pile diameter, in inches, are not recommended.

7.4 DESIGN FOR RATED ENERGY CAPACITY

As was described in Section 6.1, the rated energy capacity of a pile is defined as the energy absorbed for a maximum pile bending stress equal to 75 percent of the yield stress. The soil is treated as an elastic foundation with stiffness increasing in proportion to depth below the soil surface. Pile flexural rigidity is a function of the pile nominal diameter, D, and nominal wall thickness t, and the modulus of elasticity of the steel. Pile section modulus is a function of pile nominal diameter, D, and the net pile wall thickness, t_n, (nominal wall thickness, t, minus 0.125" corrosion allowance).

Within the embedded portion of the pile, relationships among pile shear, bending moment, deflection, slope, and soil pressure are dependent upon the "pile characteristic length," T, (which is termed the "relative stiffness factor" in NAVFAC DM-7, March, 1971). T is a function of the pile flexural rigidity and the coefficient of variation of soil modulus of horizontal subgrade reaction with depth, f. Since absorbed energy is proportionate to pile deflection, and pile deflection decreases with increasing soil stiffness, rated energy capacity, W_R, is determined for the maximum value of f at the dolphin site.

The input variables (known or assumed) are

- D, nominal pile diameter, feet
- t, nominal pile wall thickness, inches
- f_y, steel yield stress, ksi
- H, height of loading above seabed, feet
- f_{max}, maximum value of coefficient of variation of soil modulus of horizontal subgrade reaction with depth, lbs/inch^3

Enter Fig. 3 with the pile nominal diameter, D, and read T_{min} from the T_{min} curve for the pipe wall nominal thickness, t, of interest. Note that the T_{min} curves of Fig. 3 are based on f_{max} = 24 lbs/inch^3, which is appropriate for a medium sand seabed. If soil data at the specific site indicate a different value of f_{max}, compute a corrected value of T_{min}

$$\text{corrected } T_{min} = \left(\sqrt[5]{\frac{24}{f_{max}}}\right) \times (T_{min} \text{ from Fig. 3}) \qquad (4)$$

Compute $\frac{T_{min}}{H}$

Enter Fig. 4 with T_{min}/H and read the coefficient C_E from the curve for the yield stress, f_y, of interest.

Enter Fig. 5 with the pile nominal diameter, D, and read the coefficient, A_E, from the curve for the pile nominal wall thickness, t, of interest. These curves incorporate the effect of a 0.125" corrosion allowance.

The rated energy capacity, W_R, (kip-feet) is given by

$$W_R = N\, C_E\, A_E\, H \qquad (5)$$

The bases of Eqs. (4) and (5), the curves of Figs. 3, 5, and 5, are presented in an appendix.

EXAMPLE 3

For a dolphin cluster of six piles, each pile having a diameter of 4.5 feet and thickness 1.5 inches, and yield stress 50 kips/inch^2, what is the rated energy capacity for loading 70 feet above the seabed?

From Fig. 3, for D = 4.5 feet and t = 1.5 inches,

$$T_{min} = 13.6 \text{ feet}$$

$$\frac{T_{min}}{H} = \frac{13.6}{70} = 0.194$$

From Fig. 4, for $T_{min}/H = 0.194$ and $f_y = 50$ kip/inch^2,

$$C_E = 0.068 \text{ kip/inch}^2$$

From Fig. 5, for D = 4.5 feet and t = 1.5 inches,

$$A_E = 24.6 \text{ inch}^2$$

$$\therefore \quad W_R = N\, C_E\, A_E\, H$$

$$= 6(.068)(24.6)(70) = \underline{W_R = 703 \text{ kip-feet}}$$

EXAMPLE 4

Given that a dolphin must provide 180 kip-feet energy capacity under loading applied 60 feet above the seabed. How many piles are required if each pile is 3.5 feet in diameter, has a thickness of 1.0 inch, and has a yield stress of 60 kips/inch^2?

From Fig. 3, For D = 3.5 feet and t = 1.0 inch,

$$T_{min} = 10.5 \text{ feet}$$

$$\frac{T_{min}}{H} = \frac{10.5}{60} = 0.175$$

From Fig. 4, for $T_{min}/H = 0.175$ and $f_y = 60$ kip/inch^2,

$$C_E = 0.091 \text{ kip/inch}^2$$

From Fig. 5, for D = 3.5 feet and t = 1.0 inch,

$$A_E = 11.8 \text{ inch}^2$$

$$\therefore \quad W_R = N(.091)(11.8)(60) = 64.4N$$

$$\therefore \quad N = \frac{180}{64.4} = N = 2.8' \quad \text{use } \underline{N = 3 \text{ piles}}$$

7.5 MAXIMUM FORCE ON DOLPHIN

The rated force capacity of the dolphin (Section 7.3) represents the (mooring) force which may be sustained, at or near its full value, over a considerable period of time. In the energy-absorbing mode (Section 7.4) the maximum force is of brief duration, and this force is permitted to reach higher values than the rated force capacity. Thus the maximum force on the dolphin is the force producing a maximum pile bending stress equal to 75 percent of the yield stress (the basis used in determining rated energy capacity). This force is used in the determination of maximum forces resisted by inter-pile connecting elements. The maximum force on the dolphin is used in the design of fendering (not included in the contract under which these standards were prepared). If the maximum force cannot be distributed over sufficient hull area to avoid hull overstress, greater flexibility must be achieved. A different dolphin may be selected (more piles, of smaller diameters), providing the required energy capacity at smaller maximum force. Alternatively, energy-absorbing fendering may be used, thus reducing the required rated energy capacity (and the concurrent maximum force) to be provided by the dolphin piles.

The input variables (known or assumed) are

- D, nominal outside diameter of pile, feet
- t, nominal pipe wall thickness, inches
- f_y, steel yield stress, ksi
- H, height of loading above seabed, feet.

Compute t_n by Equation (1).

Compute $$M_y = 3\pi \, f_y \, D^2 \, t_n \left(1 - \frac{t_n}{4D}\right) \qquad (6)$$

Enter Fig. 3 with pile nominal diameter D, and read T_{min} from the T_{min} curve for the pipe wall thickness, t, of interest. If soil data for the specific site indicate a value of f_{max} different from f_{max} = 24 lbs/inch^3 (the basis of Fig. 3), use Eq. (4) to obtain a corrected value of T_{min}.

Compute $\frac{T_{min}}{H}$

Enter Fig. 6 with T_{min}/H and read the value of the coefficient C_M.

The maximum dolphin force, F_{max}, is given by

$$F_{max} = \frac{0.75\ M_y\ N}{C_M\ H} \tag{7}$$

EXAMPLE 5

For the dolphin cluster of EXAMPLE 3, what is the maximum force, F_{max}, corresponding to the rated energy capacity for loading 70 feet above the seabed?

$$t_n = 1.5 = 0.125 = 1.375"$$

From Eq. (6), $M_y = 3\pi(50)(4.5)^2(1.375)[1 - \frac{1.375}{4(4.5)}] = 12{,}120$ kip-feet

From Fig. 6, for $T_{min}/H = 0.194$, read $C_M = 1.065$

From Eq. (7), the maximum dolphin force is

$$F_{max} = \frac{0.75(12{,}120)(6)}{1.065(70)} = \underline{F_{max} = 732 \text{ kips}}$$

EXAMPLE 6

For the dolphin cluster of EXAMPLE 4, what is the maximum force, F_{max}, corresponding to the rated energy capacity for loading 60 feet above the seabed?

$$t_n = 1.0 - 0.125 = 0.875"$$

From Eq. (6), $M_y = 3\pi(60)(3.5)^2(.875)\ [1 - \frac{.875}{4(3.5}] = 5685$ kip-feet

From Fig. 6, for $T_{min}/H = 0.175$, read $C_M = 1.056$

From Eq. (7) the maximum force is

$$F_{max} = \frac{0.75(5685)(3)}{1.056(60)} = \underline{F_{max} = 202 \text{ kips}}$$

EXAMPLE 7

For the dolphin cluster of examples 4 and 6, for loading applied 60 feet above the seabed, the rated energy capacity and maximum force are, respectively, 3 x 64.4 = 193 kip-feet, and 202 kips. What are the corresponding rated energy capacity and maximum force when the loading is 50 feet above the seabed?

$$\frac{T_{min}}{H} = \frac{10.5}{50} = 0.21$$

From Fig. 4, for $T_{min}/H = 0.21$ and $f_y = 60$ kip/inch^2,

$$C_E = 0.100$$

$$\therefore \; W_R = N \, C_E \, A_E \, H$$

$$= 3(0.100)(11.8)(50) = \underline{W_R = 177 \text{ kip-feet}}$$

From Fig. 6, for $T_{min}/H = 0.21$, read $C_M = 1.073$

From Eq. (7), $F_{max} = \frac{0.75(5685)(3)}{1.073(50)} = \underline{F_{max} = 238 \text{ kips}}$

Thus the rated energy capacity is less (and the associated maximum dolphin force is larger) as the elevation of loading is decreased.

7.6 MAXIMUM PILE-TOP DEFLECTION AND SLOPE

The inter-pile connecting elements must be designed to accommodate the pile-top slopes and deflections. These increase with decreasing soil stiffness. Thus we compute slope and deflection based on a maximum possible value of T denoted by T_{max}. For determining T_{max} we use for f_{min} the mean value from Fig. 1 (16 lbs/inch^3) reduced by

75 percent for the possible effect of repeated loadings; i.e., $f_{min} = 4.0$ lbs/inch^3. For conservatism we determine the slope and deflection corresponding to the development of the yield bending moment, M_y.

The input variables are

- D, nominal pile outside diameter, feet
- t, nominal pile wall thickness, inches
- f_{min} = 0.25 times mean f for specific site, if soil data indicate this mean value is different from 16.0lbs/inch^3
- f_y, steel yield stress, ksi
- H, height of pile top above seabed, feet.

Enter Fig. 3 with D and read T_{max} from the T_{max} curve corresponding to the thickness, t, of interest. If soil data indicate a value of f_{min} differing from 4.0 lbs/inch^3, compute a corrected T_{max}

$$\text{corrected } T_{max} = \sqrt[5]{\frac{4.0}{f_{min}}} \times (T_{max} \text{ from Fig. 3}) \qquad (8)$$

Compute the pile flexural rigidity EI (kip-feet^2) from

$$EI = 45{,}000\,\pi\, D^3 t\left(1 - \frac{t}{4D}\right) \qquad (9)$$

Compute t_n by Eq. (1).

Compute M_y by Eq. (6).

Enter Fig. 7 with T_{max}/H and read values of the deflection coefficient C_Δ, and the slope coefficient, C_θ.

Compute maximum deflection, Δ_{max}, (feet) and maximum pile-top slope, θ_{max}, (radians) from

$$\Delta_{max} = C\,\frac{M_y H^2}{EI} \qquad (10)$$

$$\theta_{max} = C\,\frac{M_y H}{EI} \qquad (11)$$

Derivations of the above equations, and the bases for curves of Fig. 7 are presented in an appendix.

EXAMPLE 8

For the dolphin cluster of EXAMPLE 3 and 5, what are the maximum deflection and rotation at the top, 70 feet above the seabed?

From EXAMPLE 5, $M_y = 12{,}120$ kip-feet

From Fig. 3, for $D = 4.5$ feet and $t = 1.5$ inches

$$T_{max} = 19.3 \text{ feet}$$

$$\frac{T_{max}}{H} = \frac{19.3}{70} = 0.276$$

From Eq. (9)

$$EI = 45000\pi(4.5)^3(1.5)\left[1 - \frac{1.5}{4(4.5)}\right] = 17.7 \times 10^6 \text{ kip-feet}^2$$

From Fig. 7, for $T_{max}/H = 0.276$

$$C_\Delta = 1.01 \quad \text{and} \quad C_\theta = 1.00$$

From Eq. (10), $$\Delta_{max} = \frac{1.01(12{,}120)(70)^2}{17.7 \times 10^6} = \underline{3.4 \text{ feet}}$$

From Eq. (11), $$\theta_{max} = \frac{1.00(12{,}120)(70)}{17.7 \times 10^6} = \underline{0.048 \text{ radians}}$$

EXAMPLE 9

In the dolphin cluster of EXAMPLES 3, 5, and 8, pairs of piles are provided with horizontal torque arms which are chain-connected to mobilize pile torsional resistances. When the pile-top deflection and slope (determined in EXAMPLE 8) occur in the common plane of such a pile pair, what is the relative vertical deflection of the

torque arm ends? Assume each torque arm extends 6.5 feet from the pile axis.

From *EXAMPLE 8*, $\theta_{max} = 0.048$ radians.

Vertical displacement of each torque arm end (upward on one arm and downware on the other)

$$= 6.5(0.048) = 0.31 \text{ feet} = 3.7''$$

Relative vertical displacement $= 2(3.7) = \underline{7.4''}$

The chains must be installed with an initial sag sufficient to accommodate this relative vertical displacement. An initial (midlength) sag equal to one-half the anticipated maximum relative vertical displacement of the torque arm ends will suffice.

7.7 CHAIN FORCES

As described in Section 6.2, the capacity of a multiple-pile dolphin under eccentric loading can be substantially increased if inter-pile connections mobilize individual pile torsional resistances. In these standards it is recommended that this be achieved by providing pairs of piles with mating torque arms, and chain connecting them to limit relative horizontal deflections of their ends. As shown in Figs. 14 and 15, the torque arms can be incorporated in brackets which serve also as fender supports.

If loading can occur at any point in the perimeter, the most severe eccentricity can be represented by the full dolphin force, F_{max}, acting through the axis of the pile which is farthest from the dolphin axis, and in a direction perpendicular to the radial line from dolphin axis to pile axis. In many cases this assumption is too conservative, and the maximum possible eccentricity should be determined.

Denoting by e (feet) the maximum eccentricity of the applied load with respect to the dolphin axis, and denoting by Q (kip-feet) the total eccentricity moment to be resisted by all the torque-arm-connected piles, we write

$$Q = e\,F_{max} \tag{12}$$

When there are more than one pair of chain-connected pile pairs, the total dolphin torque may not be distributed equally among the pairs. Unequal distribution may result from small differences in the initial chain sags. There is no danger of overstressing individual piles, in torsion, because torsional shear stresses always are very much smaller than the pile bending stresses. However, to avoid overstress of the chains, and their connections to the torque arms, the following distributions are recommended. In each case N_c denotes the number of chain-connected piles, F_{ch} denotes the chain force, and s denotes the spacing of piles in each chain-connected pair.

$$\text{For } N_c = 2, \quad F_{ch} = \frac{Q}{N_c} \div \frac{s}{2} = \frac{Q}{s} \tag{13}$$

$$\text{For } N_c = 4, \quad F_{ch} = \frac{4}{3}\frac{Q}{N_c} \div \frac{s}{2} = \frac{2}{3}\frac{Q}{s} \tag{14}$$

$$\text{For } N_c = 6, \quad F_{ch} = \frac{3}{2}\frac{Q}{N_c} \div \frac{s}{2} = \frac{1}{2}\frac{Q}{s} \tag{15}$$

EXAMPLE 10

The dolphin of EXAMPLES 3 and 5 is comprised of six piles in a regular hexagonal arrangement. Three separate pile pairs are provided with chain-connected torque arms. The pile spacing, in each pair, is 13.5 feet. All piles are at the same radial distance from the dolphin axis, 13.5 feet, and the line of action of the applied load can fall outside the perimeter piles. In EXAMPLE 5 it was found that F_{max} = 732 kips. For what maximum force, F_{ch}, should the chains be designed?

Since the line of action can fall outside the perimeter piles, the most severe eccentricity will be assumed. Thus

$$e = 13.5 \text{ feet}$$

From Eq. (12), $Q = 13.5(732) = 9880$ kip-feet.

From Eq. (15), $F_{ch} = \frac{1}{2}\left(\frac{9880}{13.5}\right) = \underline{366 \text{ kips}}$

EXAMPLE 11

The dolphin of EXAMPLES 4 and 6 is comprised of three piles, two of which are chain connected. The pile spacing in this pair is 10.5 feet. It has been determined that the maximum eccentricity of dolphin loading is 4.6 feet. In EXAMPLE 6 it was found that F_{max} = 202. For what maximum force, F_{ch}, should the chains be designed?

$$e = 4.6 \text{ feet}$$

From Eq. (12), $Q = 4.6(202) = 929$ kip-feet

From Eq. (13), $F_{ch} = \dfrac{929}{10.5} = \underline{88 \text{ kips}}$

Fig. 8 illustrates the chain forces for a 6-pile dolphin with the most severe eccentricity of loading. The standard dolphins presented herein (Figs. 14 and 15) have been designed on the assumption that the most severe eccentricities can occur, and the chain sizes have been selected accordingly. Table I lists the ratio of chain force, F_{ch}, to maximum dolphin force, F_{max}, for dolphins similar to those presented in Figs. 14 and 15, subject to the most severe eccentricities of loading. It should be noted that many dolphins are not subject to loading at all points on their perimeters, and the most severe eccentricities may be substantially smaller than were assumed in the preparation of Table I. It will be noted that F_{ch}/F_{max} does not change greatly as the number of piles is changed.

TABLE I

RATIO OF CHAIN FORCE TO MAXIMUM DOLPHIN FORCE IN STANDARD DOLPHINS

Cluster Pattern	F_{ch}/F_{max}
2-pile cluster	0.50
3-pile (equil. triangle)	0.58
4-pile (square)	0.47
5-pile (pentagon)	0.57
6-pile (hexagon)	0.50
7-pile (hexagon plus central pile)	0.58

7.8 STRUT SYSTEMS

As was described in Section 6.3, if fender brackets are required at more than one elevation, two systems of inter-pile struts are needed. Each of these systems comprises a horizontal truss. Strut-to-pile connections consist of loose-fit horizontal pin joints which accommodate angle changes (pile axis to strut axis) associated with deflection and slope of the flexed piles. These pin joints are not on the pile axes. However, the strut axes do intersect on the pile axes, and for analysis of a strut system "truss" the truss joints can be assumed to be located on pile axes.

7.8.1 UPPER LEVEL STRUT FORCES

The upper-level truss system must hold in equilibrium the following system of forces, applied at the truss "joints"; i.e., on pile axes.

(a) At one joint the full dolphin load, F_{max}.

(b) At every joint a load F_{max}/N, where N is the total number of piles, acting in a direction opposite to the direction of load F_{max}.

(c) At each joint from which a torque arm emanates, the chain force transposed to the joint center.

The above total system of forces on the truss joints must be in equilibrium, and analysis for the strut forces to maintain this equilibrium is straightforward. Typically all struts will be designed for the maximum force developed in any of the struts. Depending upon the cluster pattern, and upon the location of chain-connected pile pairs, it may not be obvious which truss joint must be subjected to the force F_{max} in order to produce the maximum strut force. This uncertainty may necessitate the investigation of max strut forces for each of several choices of the loaded joint.

For any joint to which the dolphin force is applied it is apparent that the maximum dolphin torque (and, thus, the maximum chain force) occurs when the applied load is perpendicular to the radial line from dolphin axis to pile axis. This orientation does not necessarily produce

the largest strut forces. For example, in a two-pile cluster maximum strut force occurs when the applied load acts along the radial line from dolphin axis to pile axis. Thus, more generally, for each pile to which the dolphin load is applied, one must vary the direction of loading to determine the maximum possible strut force.

Table II lists the ratios of strut forces, F_{st}, to maximum dolphin force, F_{max}, for dolphins similar to the standard dolphins presented in Figs. 14 and 15. These ratios apply only to (regular polygonal) configuration of the standard dolphin with loading possible at any point on the perimeter.

TABLE II

RATIO OF UPPER-LEVEL MAXIMUM STRUT FORCE TO MAXIMUM DOLPHIN FORCE IN STANDARD DOLPHINS

Cluster Pattern	F_{st}/F_{max}
2-pile	0.50
3-pile (triangle)	0.67
4-pile (square)	0.71
5-pile (pentagon)	0.93
6-pile (hexagon)	1.20

7.8.2 LOWER-LEVEL STRUT FORCES

The lower-level system of inter-pile struts is of the same geometrical form as the upper-level system. However, the loading applied to the lower strut system differs from that applied to the upper-level system. The difference arises (a) from the fact that the dolphin force, F_{max}, is larger for loading applied at the lower level, and (b) from the fact that the piles are not chain-connected at the lower level.

The lower-level strut system must hold in equilibrium the following system of forces, applied at the truss "joints"; i.e., on pile axes.

(a) At one joint the full dolphin load, F_{max}.

(b) At every joint, a load F_{max}/N, where N is the total number of piles, acting in a direction opposite to the direction of load F_{max}.

(c) At every joint, a load perpendicular to the radial line from dolphin axis to pile axis. These loads together produce a moment about the dolphin axis which is equal and opposite to the eccentricity moment of the dolphin load F_{max}. The magnitude of each load is proportionate to the radial distance from dolphin axis to pile axis. Thus if the perimeter piles are in a regular polygonal pattern, each load is equal to the eccentricity moment divided by the product of the radial distance and the number of perimeter piles.

Table III lists the ratios of strut forces, F_{st}, to maximum dolphin force, F_{max}, for dolphins similar to the standard dolphins presented in Figs. 14 and 15. These ratios apply only to the (regular polygonal) configuration of the standard dolphin with loading possible at any point on the perimeter.

TABLE III

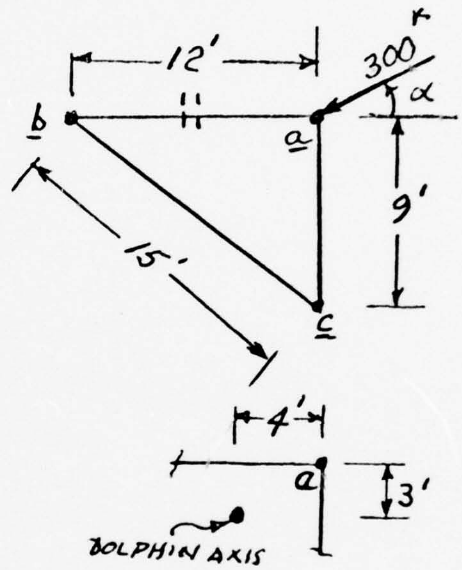
RATIO OF LOWER-LEVEL MAXIMUM STRUT FORCE TO MAXIMUM DOLPHIN FORCE IN STANDARD DOLPHINS

Cluster Pattern	F_{st}/F_{max}
2-pile	0.50
3-pile (triangle)	0.58
4-pile (square)	0.69
5-pile (pentagon)	0.77
6-pile (hexagon)	0.92

7.8.3 CHAIN FORCES AND STRUT FORCES IN DOLPHIN OF IRREGULAR CONFIGURATION

This Section illustrates steps in the determination of forces in the inter-pile connections for a multi-pile dolphin which is not one of the standard configurations. There is, of course, an infinite number of possible configurations, and only the method of analysis can be illustrated. Since dolphins comprised of a small number of large piles are to be preferred, a three-pile dolphin will be treated in the examples.

EXAMPLE 12



Consider a 3-pile dolphin with axes of the piles, a, b, and c located as shown. At the upper level, the maximum dolphin force, F_{max}, has been found to be 300 kips. Fenders are mounted on pile a only. It has been determined that the line of action of the dolphin force, defined by α, can lie anywhere within the range $\alpha = 0$ to $\alpha = 90°$. The upper level inter-pile connections are comprised of struts ab, bc, and cd, and chain-linked torque arms between a and b. Determine the maximum chain force, F_{ch}, and maximum strut force, F_{st}.

The dolphin axis is located at the centroid of the pile group; i.e., 3 feet from ab and 4 feet from ac. The dolphin torque Q (counterclockwise) is given by

$$Q = 3(300 \cos \alpha) - 4(300 \sin \alpha)$$
$$= 900 \cos \alpha - 1200 \sin \alpha \qquad \text{kip-feet}$$

Since piles a and b are connected by chain-linked torque arms, these two piles will be assumed to resist all of the dolphin torque. From Eq. (13) the chain force is given by

$$F_{ch} = \frac{Q}{s} = \frac{1}{12}[900 \cos \alpha - 1200 \sin \alpha]$$
$$= 75 \cos \alpha - 100 \sin \alpha \qquad \text{kips.}$$

Clearly, within the given range of α, F_{ch} reaches maxima of 75 kips (for counterclockwise dolphin torque) and -100 kips (for clockwise torque). While two different chain sizes could be used, it probably is more practical to design both chains, their connections, and the torque arms, for F_{ch} = 100 kips in each of the two directions.

Since F_{max} (=300 kips) is shared equally by the three piles, truss joint a experiences a net force (applied force minus pile resisting shear) equal to 2/3 F_{max}, in the direction of applied loading. Piles b and c each experience a force 1/3 F_{max} (the pile shear) opposite in direction to the applied loading. In addition, piles a and b each experience the chain force, F_{ch}. The combined joint forces will be

at a, directed toward b, $\frac{2}{3}(300 \cos\alpha) = 200 \cos\alpha$

at a, directed toward c, $\frac{2}{3}(300 \sin\alpha) + (75 \cos\alpha - 100 \sin\alpha)$
$= 100 \sin\alpha + 75 \cos\alpha$

at b, directed toward a, $\frac{1}{3}(300 \cos\alpha) = 100 \cos\alpha$

at b, directed parallel to a c, $\frac{1}{3}(300 \sin\alpha) + (75 \cos\alpha - 100 \sin\alpha)$
$= 75 \cos\alpha$

at c, directed parallel to a b, $\frac{1}{3}(300 \cos\alpha) = 100 \cos\alpha$

at c, directed toward a, $\frac{1}{3}(300 \sin\alpha) = 100 \sin\alpha$

Analysis of the simple truss comprised of the three struts is straightforward. It leads to the following strut forces

$$F_{ab} = -200 \cos\alpha \quad \text{kips}$$
$$F_{ac} = -75 \cos\alpha - 100 \sin\alpha, \quad \text{kips}$$
$$F_{bc} = +125 \cos\alpha \quad \text{kips}$$

The largest strut force is seen to occur in a b, and to equal 200 kips, at $\alpha = 0$.

Conclusion, design struts for F_{st} = 200 kips.

EXAMPLE 13

The lower-level inter-pile connections, for the dolphin of EXAMPLE 12, are comprised only of struts; i.e., chain-linked torque arms are in the upper-level system and are omitted from the lower-level system. The maximum dolphin force which can be applied at the lower level has been determined to be 350 kips. It is applied at the same pile, a, and its line of action is in the same range of angle α as was the upper-level dolphin force. For what maximum force should the lower-level struts be designed?

Maximum dolphin torque Q (counterclockwise) is given by

$$Q = 3(350 \cos\alpha) - 4(350 \sin\alpha)$$
$$= 1050 \cos\alpha - 1400 \sin\alpha \qquad \text{kip feet}$$

The sum of the squares of the radial distances from dolphin axis to pile axes is $(5)^2 + (7.2)^2 + (8.6)^2 = 150$. Each pile will contribute a force component, normal to the radial line from dolphin axis to pile axis, and proportionate to the radial distance, thus balancing the dolphin torque Q. These forces are

	FORCE	TORQUE
For pile a, $\frac{5}{150}Q =$	.037 Q;	5(.033Q) = .17Q
For pile b, $\frac{8.6Q}{150} =$	.057 Q;	8.6(.057Q) = .48Q
For pile c, $\frac{7.2Q}{150} =$	.048 Q;	7.2(.048Q) = .35Q
		1.00Q

It is convenient to resolve the above joint forces into components, parallel to a b and a c, and to combine them with the direct (non-torque-related) force components. The final joint force components thus are,

at a, directed toward b,

$$-\frac{3}{5}(.033)(1050 \cos\alpha - 1400 \sin\alpha) + \frac{2}{3}(350 \cos\alpha) = 212 \cos\alpha + 28 \sin\alpha$$

at a, directed toward c,

$$\frac{4}{5}(.033)(1050 \cos\alpha - 1400 \sin\alpha) + \frac{2}{3}(350 \sin\alpha) = 28 \cos\alpha + 196 \sin\alpha$$

at b, directed toward a,

$$\frac{3}{8.6}(.057)(1050\cos\alpha - 1400\sin\alpha) + \frac{1}{3}(350\cos\alpha) = 138\cos\alpha - 28\sin\alpha$$

at b, parallel to a c,

$$\frac{8}{8.6}(.057)(1050\cos\alpha - 1400\sin\alpha) + \frac{1}{3}(350\sin\alpha) = 56\cos\alpha + 42\sin\alpha$$

at c, directed toward a,

$$-\frac{4}{7.2}(.048)(1050\cos\alpha - 1400\sin\alpha) + \frac{1}{3}(350\sin\alpha) = -28\cos\alpha + 154\sin\alpha$$

at c, parallel to a b,

$$-\frac{6}{7.2}(.048)(1050\cos\alpha - 1400\sin\alpha) + \frac{1}{3}(350\cos\alpha) = 74\cos\alpha + 56\sin\alpha$$

For the above set of joint forces the strut forces are readily obtained; they are

$$F_{ab} = (138\cos\alpha - 28\sin\alpha) + \frac{4}{3}(56\cos\alpha + 42\sin\alpha)$$
$$= 212\cos\alpha + 28\sin\alpha \qquad \text{kips, compression}$$

$$F_{ac} = (-28\cos\alpha + 154\sin\alpha) + \frac{3}{4}(74\cos\alpha + 56\sin\alpha)$$
$$= 28\cos\alpha + 196\sin\alpha \qquad \text{kips, compression}$$

$$F_{bc} = \frac{5}{3}(56\cos\alpha + 42\sin\alpha)$$
$$= 93\cos\alpha + 70\sin\alpha \qquad \text{kips, tension}$$

It is apparent that the largest strut force occurs in strut a b. Thus

$$F_{st} = \sqrt{(212)^2 + (28)^2} = \underline{F_{st} = 214 \text{ kips}}, \text{ compression}$$

All lower-level struts may be designed for 214 kips compression force. However, because its force is much smaller, and in tension, it may be reasonable to design strut bc for

$$\sqrt{(93)^2 + (70)^2} = \underline{116 \text{ kips,}} \text{ tension}$$

7.9 STEEL GRADES AND STRESS LEVELS

7.9.1 PIPE PILES

If the function of a dolphin is solely to provide a mooring point, the emphasis is on strength. For such an application large diameter piles, of structural grade carbon steel, may be an economical choice. The relatively low yield strength can be offset by the use of fairly large wall thickness in the zone of high bending moment, and the thickness can be reduced, in steps, toward the top and bottom of the piles.

If the functions of the dolphin include absorption of energy, there is a very strong incentive to use higher strength steel. This incentive derives from the fact that elastic energy increases in proportion to the second power of the stress level. Thus the extra cost of high strength steel, and the additional care required to achieve satisfactory welded joints, may be more than offset by very large increases in elastic energy capacity. It should be noted that, while high strength steel pile dolphins have not been common in the United States, they have a history of successful applications in Europe extending over more than two decades.

In these standards it is recommended that steel having a yield strength of the order of 60 ksi be used in high bending moment regions of dolphin piles required to function in an energy-absorbing mode. The bending stresses suitable for this steel grade are the highest judged advisable without introducing special measures, such as for example sleeves, to limit soil stresses. For the standard dolphins presented herein, A572 appears to be a satisfactory choice, but there are satisfactory alternative steels. It must be noted that the steel, and the welding procedures (including pre-heat, full-penetration flush ground welds, and possibly post-weld heat treatment) must be selected to minimize the possibility of brittle fracture.

In the energy-absorbing mode it is recommended that the pile be designed for a maximum bending stress equal to 75 percent of the yield

stress. While this may seem to be a high design stress, it is justified by several factors. First, stress is computed on the basis of a section modulus conservatively reduced for the effects of corrosion. Second, the rated energy condition is computed on the basis of maximum soil stiffness; at a (more probable) lower soil stiffness the rated energy will be developed at a lower value of bending stress. Third, maximum bending stress is unlikely to be experienced more than a few thousand times during the service life of the dolphin. Fourth, the regions of high bending stress are underwater; therefore the service temperature cannot be below 30° F, which is not severe.

In those (upper and lower) portions of the pile where the bending moments are equal to, or less than, fifty percent of the maximum bending moment, a structural grade carbon steel should be used. Minimum thicknesses in these zones are not service stress dependent. At the lower end resistance to damage during pile driving is the principal consideration. At the top, corrosion is the controlling factor.

7.9.2 INTER-PILE CONNECTING ELEMENTS

Torque arms, fender brackets, struts, and their connecting details are in a severely corrosive environment. In some geographic regions these components also may experience low service temperatures. Structural grade steels are recommended and design stress levels should provide a factor of safety of at least 2.0 with respect to yield.

7.9.3 CHAINS

When chain-linked, torque arms are used to mobilize individual pile torsional resistances, the use of the highest strength grade is advantageous. Stud-link, forged steel anchor chain (ABS or Lloyd'd class), designed for maximum service stress equal to fifty percent of the proof stress, is recommended.

7.10 PILE SPACING AND EMBEDMENT DEPTHS

Center-to-center pile spacings should not be less than three pile diameters. Embedment lengths below the top of the sand stratum are equal to 4 T_{min}, but not less than 3 T_{max}. Values of T_{min} and T_{max} given in Fig. 3 are based upon the conservative values f_{max} = 24 lbs/inch^3 and f_{min} = 0.25(16) = 4.0 lb/inch^3. The value of f_{min} represents the mean f for the range of soil considered, reduced 75 percent for the effect of repeated loading. These values of f_{max} and f_{min} should be used unless soil data for a specific site dictate more conservative (i.e., lower) values.

8.0 STANDARD DOLPHINS

The contract under which these standards were prepared required six standard dolphins, designed to accept loading at any point on their perimeters, providing a range of capacities in water depths of 40, 50, 60, and 70 feet. In the course of developing the dolphins presented herein, it became apparent that single-pile dolphins would satisfy the specified conditions of loading over a wide range of capacities. Such single-pile dolphins would have utilized piles ranging in diameter from 3 to 6 feet. In a few cases, requiring very large capacity, two or three of the large piles would be connected in a single dolphin. The single-pile dolphin is, of course, the ultimate in simplicity, minimizing both fabrication and maintenance costs.

In accordance with a decision of the Naval Facilities Engineering Command, the standard dolphins presented herein are comprised either of 36" diameter piles or 48" diameter piles. See Figs. 14 and 15. At the request of the Navy, these standard designs have been supplemented by a table of capacities of large single piles (Fig. 13) in the water depths of interest.

In the standard dolphins the pile wall thicknesses are held constant at 0.75". This thickness is adequate from the viewpoint of corrosion, in the salt spray zone where pile stresses are low, and is adequate to prevent damage during driving. Steels of two strength grades (60 ksi and

and 36 ksi) are used in each pile. Because of the constant, uniform, wall thickness, there will be no stress concentrations due to step changes in thickness. However, the girth seams should be full penetration butt welds, flush ground to minimize stress concentration. Within the region of high bending moments, consideration should be given to preheat and post-weld heat treatment appropriate to the specific steels selected.

Because the standard dolphins are required to accept loading at all points on the perimeter, brackets are provided which serve as fender supports as well as torque arms and, in addition, incorporate the terminals for the inter-pile struts. These brackets are designed as weldments, to be fabricated on shore. Typically, plate thicknesses and weld sizes are not governed by service load stresses, but by corrosion considerations.

Pairs of piles are chain linked, through the torque arms, at the upper level only. Inter-pile struts form horizontal trusses at both upper and lower fender bracket elevations. At some sites the tidal range may be small enough to permit fender mounting at a single elevation. In such cases the brackets, struts, and chains shown for the upper level should be used at the single level of fender mounting. Where the tidal range is very large it may be necessary to provide fender supports at three elevations. In such cases brackets with struts should be installed at the lowest of the three elevations. No inter-pile struts need be provided at the intermediate-level fender brackets.

The symmetrical pile clusters used in the standard dolphins are suitable for underwater geometries unconstrained by adjacent structures or by limitations in the performance of ships in channels, maneuvering areas, moorings, berths, or by the ships' characteristics. When any of these constraints are present, it may be necessary to alter the dolphin configuration. The capacities of a dolphin of special configuration will be identical with the capacities tabulated on Fig. 14, provided that the number of piles and pile sizes are the same as those specified for the standard dolphin. However, if the configuration differs significantly from the standard symmetrical form shown, the inter-pile forces should be analyzed, and the chain and strut designs modified accordingly.

The pin-ended struts for the standard dolphins are tubular elements. Other structural shapes, of equal capacity, may be substituted. However, the tubular form facilitates adjustability of strut length, simplifying installation, and presents the minimum surface area exposed to corrosion.

If ships will contact the dolphin only on a limited portion of the perimeter, the brackets shown can be simplified accordingly. Modification of the shape of the brackets may be indicated by results of a forthcoming EI project entitled, "Fendering for Structural Steel Dolphins." The scope of the contract under which the present standards were prepared did not include fendering.

9.0 INSTALLATION

For the cohesionless soil to which these dolphins are applicable, it is not anticipated that there normally will be any difficulty in driving the piles to the required embedment depths. If necessary, internal jetting may be used, but external jetting is not recommended.

To facilitate installation of the interpile framing, care must be taken that the individual piles are as nearly plumb as possible. Specifically, pairs of piles which are to be chain connected should lie as nearly as possible in a common plane. Slight tilt of that common plane from vertical, or slight departure of the two pile axes from parallelism within the common plane, are less serious.

The second pile of any chain-connected pair should be driven with the aid of templates to assure its proper center-to-center spacing from the first pile of the pair. However, if this spacing is achieved to a tolerance of ± 6" from nominal there will be no difficulty in subsequent steps. Control of the spacing is necessary to assure that the upper-level, mating torque arms will be properly spaced from each other Each lower-level strut connecting a pair of piles which is subsequently to be chain-connected is of a fixed length. The two piles may be either pulled together, or jacked apart, by forces applied at the upper level, to adjust the spacing so that the fixed-length lower-level strut can be pinned in

place. This procedure assures proper spacing, at the upper level, for later installation of chains.

For the "stiffest case" (48" diameter piles in minimum water depth), jacking or pulling to correct the pile spacing by, up to, 6" will not produce excessive initial bending stress in the piles. At greater water depths, for which greater corrections of spacing may be required, larger corrections are possible without overstress. The 36" diameter piles are susceptible to even larger spacing corrections without overstress.

After lower-level struts are in place, chains connecting the upper-level torque arms are installed. Each chain length must be adjusted to provide a mid-length sag to accommodate the maximum anticipated relative vertical displacement of the connected torque arm ends. A 3" to 6" sag will be satisfactory for all of the standard dolphins presented on Figs. 14 and 15. It is less important what sag is introduced (from 3" to 6") than that all chains in a dolphin be adjusted as closely as possible to the same sag.

After installation of the chains the adjustable-length upper-level strut can be installed, and its mating tubes welded to eliminate the length adjustability.

Each pair of piles which are to be chain-connected should be driven and connected by both upper and lower struts before either pile is connected to other piles in the cluster. When all such pile pairs have been installed and connected, the non-chain-connected piles should be driven and the remaining upper and lower struts installed.

Cover plates, with bollards or other special mooring elements, are welded to the pile tops; and a fendering system as required (not within the contract under which these standards were prepared) bolted to the brackets, completing the dolphin installation.

REFERENCES

(1) Terzaghi, Karl, "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, Vol. 5, No. 4, December 1955.

(2) Broms, Benet B., "Lateral Resistance of Piles in Cohesionless Soils," Journal of the Soil Mechanics and Foundations Division, Proc. of the American Society of Civil Engineers, May, 1964.

(3) Design Manual - Soil Mechanics, Foundations, and Earth Structures, NAVFAC DM-7, March 1971.

APPENDIX - A

DERIVATIONS OF EQUATIONS AND CHARTS

A.1 YIELD MOMENT, M_y

For a tube of outside radius, r_o (inches), and inside radius, r_i (inches), the moment of inertia, I (inch^4), is given by

$$I = \frac{\pi}{4}(r_o^4 - r_i^4) \tag{A1}$$

For a wall thickness, t (inches),

$$r_i = r_o - t \tag{A2}$$

$$I = \frac{\pi}{4}[r_o^4 - (r_o - t)^4] \tag{A3}$$

For r_o much larger than t, we can drop the last two terms in the expansion $(r_o - t)^4$. Then

$$I = \frac{\pi}{4}[r_o^4 - (r_o^4 - 4r_o^3 t + 6r_o^2 t^2)]$$

$$= \frac{\pi}{4}(4r_o^3 t - 6r_o^2 t^2) \tag{A4}$$

The section modulus is given by

$$S = \frac{I}{r_o} = \pi r_o^2 t \left(1 - 1.5\frac{t}{r_o}\right) \tag{A5}$$

The outside radius of a pile is so little reduced by corrosion that, for a nominal pipe outside diameter D (feet) we may write

$$r_o = \frac{1}{2}(12D) = 6D \qquad \text{inches} \tag{A6}$$

The net wall thickness t_n (inches), after 0.125" corrosion allowance is

$$t_n = t - 0.125'' \tag{A7}$$

Substituting from (A6) into (A5), and taking t_n in place of t, the section modulus, ($inch^3$), is given by

$$S_n = \pi (6D)^2 \ t_n \ (1 - \frac{1.5t_n}{6D})$$

$$= 36\pi D^2 t_n \ (1 - \frac{t_n}{4D}) \qquad inch^3 \qquad (A8)$$

$$M_y \text{ (kip-feet)} = \frac{1}{12} M_y \text{ (kip-inch)}$$

$$= \frac{1}{12} \ [S_n(inch^3) \times f_y \ (ksi)]$$

$$= \frac{1}{12} \ [36 \ D^2 \ t_n \ (1 - \frac{t_n}{4D})] \ f_y$$

$$= 3\pi f_y \ D^2 t_n \ (1 - \frac{t_n}{4D}) \qquad (A9)$$

Eq. (2) is obtained directly from Eq. (A9), and Eq. (6) is identical with Eq. (A9).

A.2 PILE RATED FORCE, F_R

As suggested by Broms,[2] assume a hinge in the pile at a depth z_o (feet), and passive soil pressure above the hinge. The soil pressure (kips per foot of pile length) is assumed to be

$$p = 3DK_p \frac{\gamma_s}{1000} z \qquad kips/ft \qquad (A10)$$

where, D = pile diameter, feet
K_p = soil passive pressure coefficient
γ_s = submerged weight of soil, lbs/ft^3
z = depth, feet

Let F_y (kips) be the force, at a distance H (feet) above the seabed, which develops the hinge condition. Since the pile shear is zero at the hinge, horizontal force equilibrium requires that

$$F_y = \int_0^{z_o} p dz = \frac{1}{2} (3Dk_p \frac{\gamma_s}{1000}) z_o^2 \qquad (A11)$$

The soil pressure varies linearly with depth, and the resultant soil force (equal and opposite to F_y) is at a distance 2/3 z_o below the seabed. Moment equilibrium requires that

$$M_y = F_y (H + \frac{2}{3} z_o) \qquad (A12)$$

Dividing Eq. (A11) by D, and solving for z_o,

$$z_o = \sqrt{\frac{F_y/D}{1.5K_p\gamma_s/1000}} \qquad (A13)$$

Dividing Eq. (A12) by D, and substituting from (A13) into (A12)

$$\frac{M_y}{D} = \frac{F_y}{D} \left[H + \frac{2}{3} \sqrt{\frac{F_y D}{1.5K_p\gamma_s/1000}} \right] \qquad (A14)$$

The pile rated force, F_R, is defined as 0.5 F_y. K_p is taken, conservatively, as 3.0, and γ_s is taken as 50 lbs/ft^3. Substitution into Eq. (A14) gives

$$\frac{M_y}{D} = (\frac{2F_R}{D}) \left[H + \frac{2}{3} \sqrt{\frac{(2F_R/D)}{0.225}} \right] \qquad (A15)$$

Eq. (A15) is the basis for the curves given in Fig. 2.

A.3 PILE CHARACTERISTIC LENGTH, T

Note that the pile characteristic length is termed the "relative stiffness factor" in NAVFAC DM7, March, 1971. T has the units length (feet, as used in this report.)

$$T \text{ (feet)} = \frac{1}{12} \sqrt[5]{\frac{E(\text{lbs/inch}^2) I(\text{inch})^4}{f(\text{lbs/inch}^3)}} \qquad (A16)$$

Substituting from Eq. (A6) into Eq. (A4)

$$I = \pi r_o^3 t \left(1 - 1.5 \frac{t}{r_o}\right)$$

$$= 216\pi D^3 t \left(1 - \frac{t}{4D}\right), \quad \text{inch}^4 \qquad \text{(A17)}$$

The curves of Fig. 3 are based on Eq. (A16) with $E = 3 \times 10^7$ psi, and I as given by Eq. (A17). Note that the thickness, t, is <u>not</u> reduced by a corrosion allowance. The curves for T_{min} are based on $f = f_{max} = 24$ lbs/inch^3. The curves for T_{max} are based on $f = f_{min} = 4$ lbs/inch^3. f_{min} is the mean f (from Fig. 1) reduced 75 percent for the effect of repeated loading.

A.4 PILE-TOP SLOPE AND DEFLECTION

From Figs. 13-4 and 13-5 of NAVFAC DM7, March, 1971, we obtain coefficients of pile deflection and pile slope at the seabed. Using these coefficients, and assuming a constant pile EI (kip-ft^2) the deflection and slope of the pile at a point of loading H(feet) above the seabed, due to a force F (kips), are

$$\Delta = \frac{2.44\ FT^3}{EI} + \frac{3.25\ FT^2H}{EI} + \frac{1.75\ FTH^2}{EI} + \frac{FH^3}{3EI}$$

$$= \frac{FH^3}{EI}\left[2.44\left(\frac{T}{H}\right)^3 + 3.25\left(\frac{T}{H}\right)^2 + 1.75\left(\frac{T}{H}\right) + .333\right] \qquad \text{(A18)}$$

$$\theta = \frac{1.62\ FT^2}{EI} + \frac{1.75\ FTH}{EI} + \frac{FH^2}{2EI}$$

$$= \frac{FH^2}{EI}\left[1.62\left(\frac{T}{H}\right)^2 + 1.75\left(\frac{T}{H}\right) + 0.500\right] \qquad \text{(A19)}$$

Pile bending moment coefficients, for the separate effects of pile shear and seabed pile moment, are given on Fig. 13-4 of NAVFAC DM7, March 1971. Using these curves, one obtains, for each ratio T/H, a coefficient of pile bending moment, as a function of dimensionless depth ratio (depth

divided by T). The maximum value of this coefficient, denoted by C_M, is charted vs. T/H on Fig. 6. The maximum bending moment, M, is given by

$$M = C_M F H \tag{A20}$$

To express pile-top slope and deflection as functions of M, substitute Eq. (A20) into Eqs. (A18) and (A19).

$$\Delta = \frac{MH^2}{C_M EI} \left[2.44 \left(\frac{T}{H}\right)^3 + 3.25 \left(\frac{T}{H}\right)^2 + 1.75 \left(\frac{T}{H}\right) + .333\right] \tag{A21}$$

$$\theta = \frac{MH}{C_M EI} \left[1.62 \left(\frac{T}{H}\right)^2 + 1.75 \left(\frac{T}{H}\right) + 0.500\right] \tag{A22}$$

The coefficients C_Δ and C_θ, used in Eqs (10) and (11), and plotted on Fig. 7, are given by

$$C_\Delta = \frac{1}{C_M} \left[2.44 \left(\frac{T}{H}\right)^3 + 3.25 \left(\frac{T}{H}\right)^2 + 1.75 \left(\frac{T}{H}\right) + .333\right] \tag{A23}$$

$$C_\theta = \frac{1}{C_M} \left[1.62 \left(\frac{T}{H}\right)^2 + 1.75 \left(\frac{T}{H}\right) + .500\right] \tag{A24}$$

In applying Eqs. (10) and (11), EI is required in units of kip-feet^2. Using E = 30,000 ksi, and I (inch^4) from Eq. (A17),

$$\begin{aligned} EI\ (\text{kip-ft}^2) &= \frac{1}{144}\,(30{,}000)\left[216\pi D^3 t\ \left(1 - \frac{t}{4D}\right)\right] \\ &= 45{,}000\ \pi D^3 t\ \left(1 - \frac{t}{4D}\right) \end{aligned} \tag{A25}$$

Eq. (9) is identical with Eq. (A25). Note that t is the pile wall thickness, unreduced by a corrosion allowance. When it is desired to know the maximum values of pile-top slope and deflection, M is (conservatively) assumed equal to M_y. Further, Fig. 7 is entered with T_{max}/H.

A.5 PILE RATED ELASTIC ENERGY, W_R

The elastic energy absorbed by a pile is equal to the work done by the applied load. For deflection proportionate to load, the energy thus is 0.5 times load times deflection.

$$W = \frac{F\Delta}{2} = \frac{F}{2}\left(\frac{MH^2C_\Delta}{EI}\right) \qquad \text{(A26)}$$

Substituting Eq. (A20) into Eq. (A26)

$$W = \frac{1}{2}\left(\frac{M}{HC_m}\right)\left(\frac{MH^2C_\Delta}{EI}\right)$$

$$= \frac{1}{2}\,\frac{M^2H}{EI}\,\frac{C_\Delta}{C_M} \qquad \text{(A27)}$$

The Rated Energy, as defined in these standards, is the energy corresponding to $M = 0.75M_y$, where M_y (kip-feet) is given by Eq. (A9). I (inch^4) is given by Eq. (A17), and E = 30,000 (ksi). Substituting into Eq. (A27), and converting E and I into foot units, the Rated Energy, W_R (kip-ft), is

$$W_R = \frac{\frac{1}{2}\,\frac{C_\Delta}{C_M}\,H\,[0.75\,f_y\pi 3\,D^2\,t_n(1 - \frac{t_n}{4D})]^2}{[144(30{,}000)][\frac{216}{(144)^2}\,\pi D^3 t\,(1 - \frac{t}{4D})]}$$

$$= \left\{\frac{2(.75)^2 f_y^2}{30{,}000}\,\frac{C_\Delta}{C_M}\right\}\left\{\frac{1.5\pi D t_n^2\,(1 - t_n/4D)}{t(1 - t/4D)}\right\} H \qquad \text{(A28)}$$

$$= C_E\,A_E\,H \qquad \text{(A29)}$$

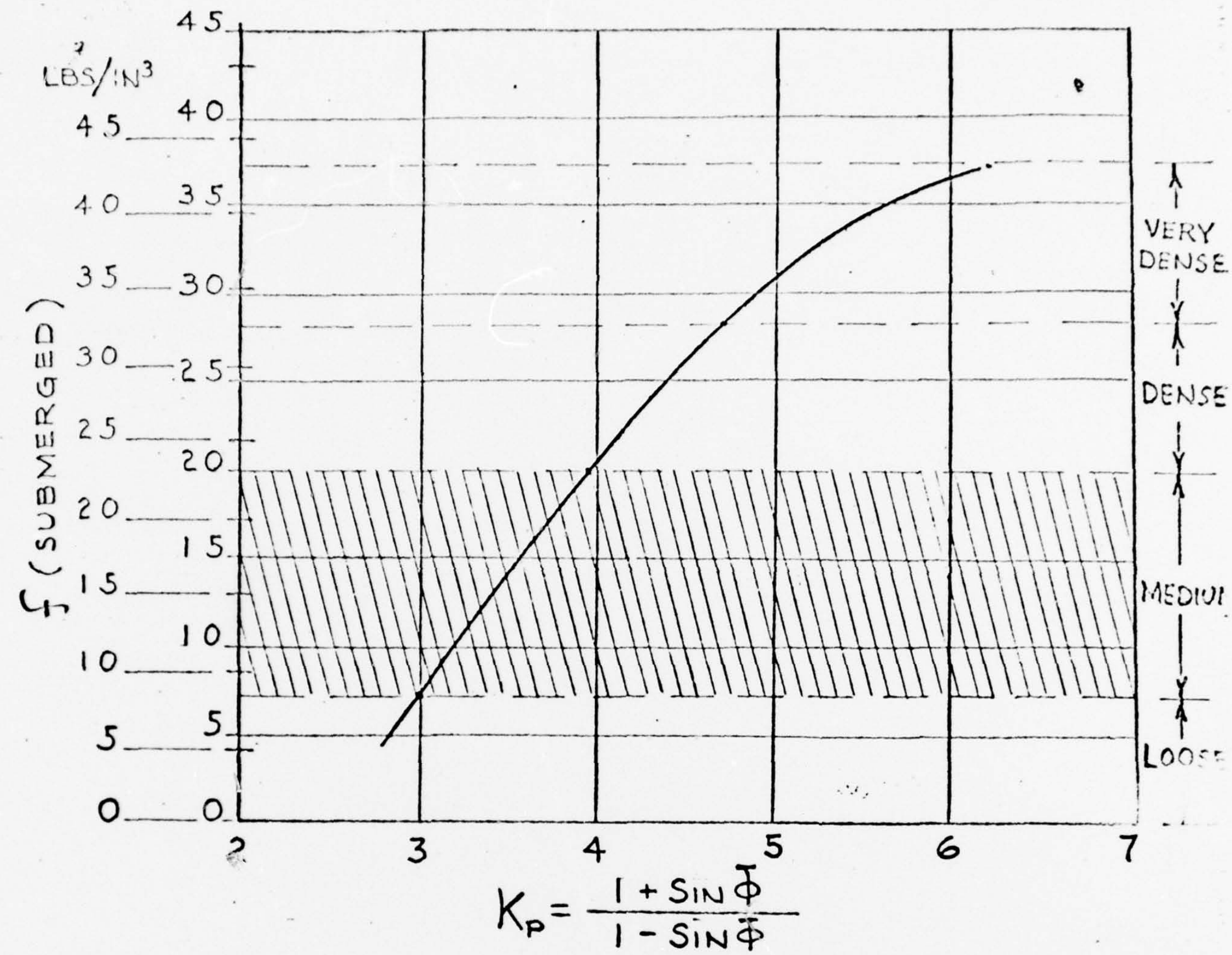
Note that C_E has the dimensions ksi, and A_E has the dimensions inch^2, although D is used in foot units.

Since the dolphin piles typically are constrained to share the load equally, an N-pile dolphin will absorb an amount of energy N times that given by Eq. (A29).

$$W_R = NC_E\,A_E\,H \quad \text{kip-feet} \qquad \text{(A30)}$$

which is identical with Eq. (5).

Since the minimum deflection, and minimum energy, corresponds to $T = T_{min}$, the factor C_E was evaluated with C_Δ and C_M determined for T_{min}/H, which is conservative. Curves of C_E and A_E are provided on Figs. 4 and 5 respectively.



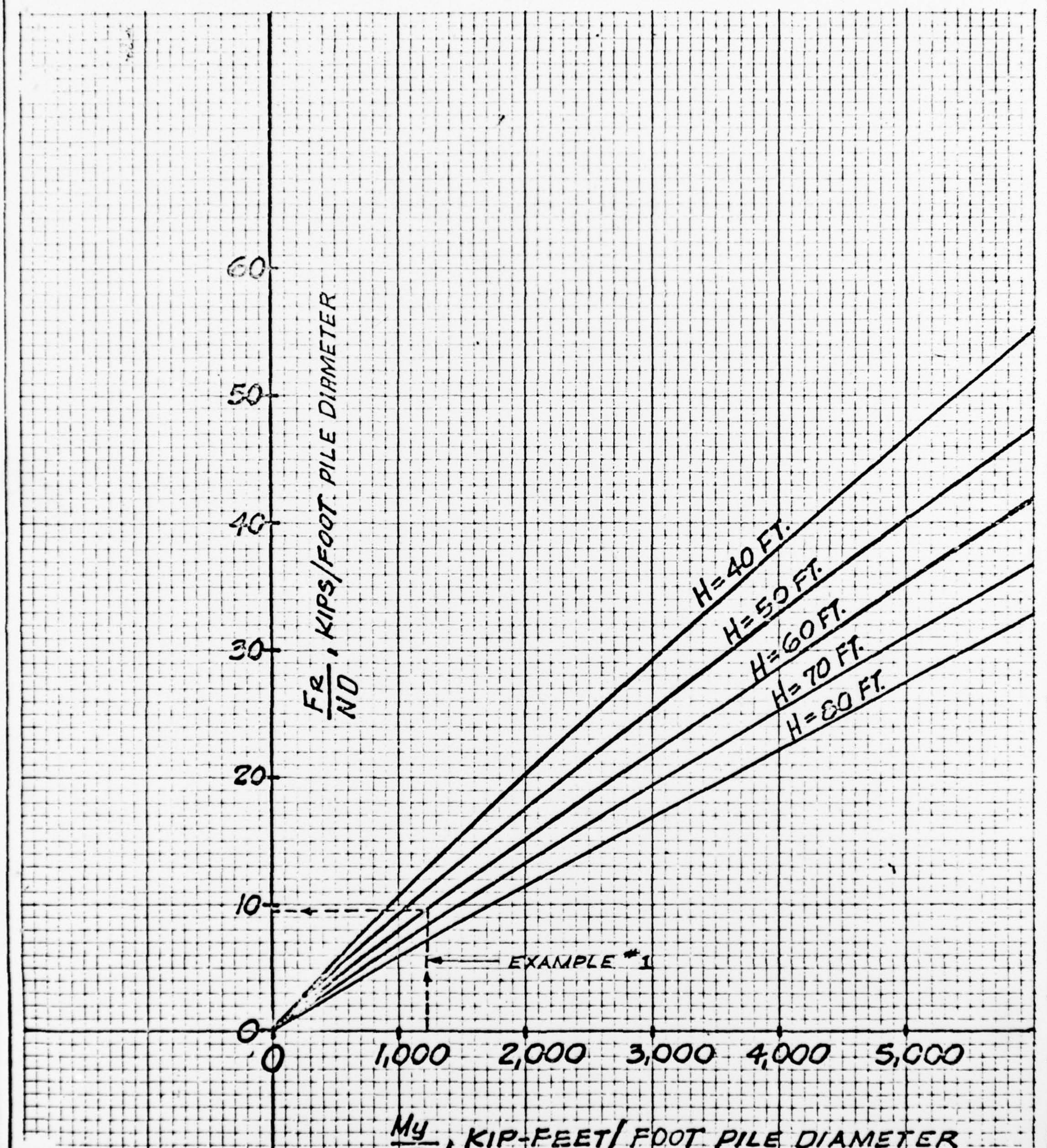
HORIZONTAL SUBGRADE REACTION COEFFICIENT, f, AND COEFFICIENT OF PASSIVE RESISTANCE, K_p

NOTE:

VALUES OF f (SUBMERGED) WERE OBTAINED BY REDUCING THE VALUES OF f FOR COARSE GRAINED SOILS FROM FIG. 11-8 OF NAVFAC DM-7 BY 60% TO ACCOUNT FOR THE EFFECTS OF SUBMERGENCE.

FIG 1

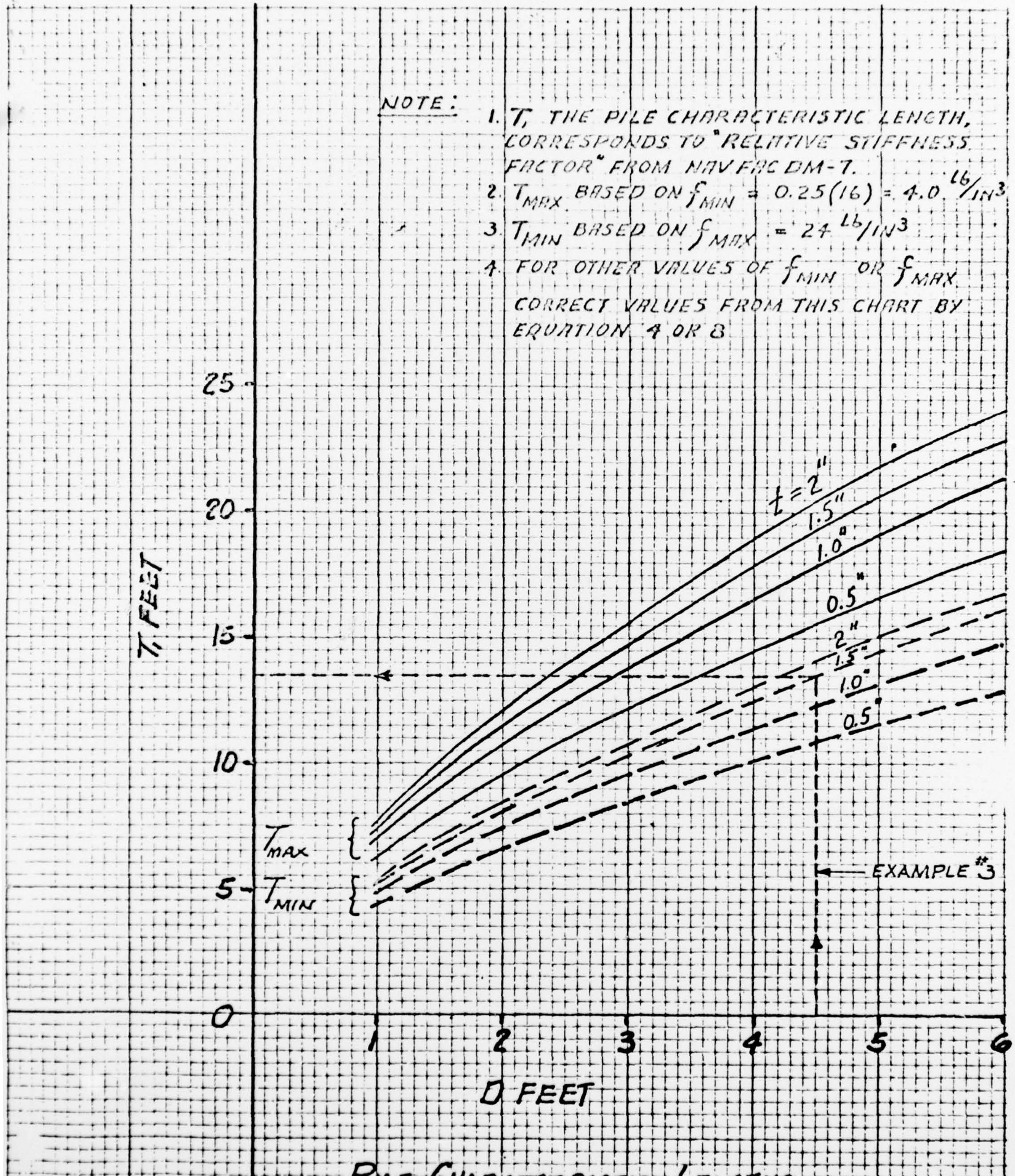
OF 15



DOLPHIN RATED FORCE
VS. PILE YIELD MOMENT

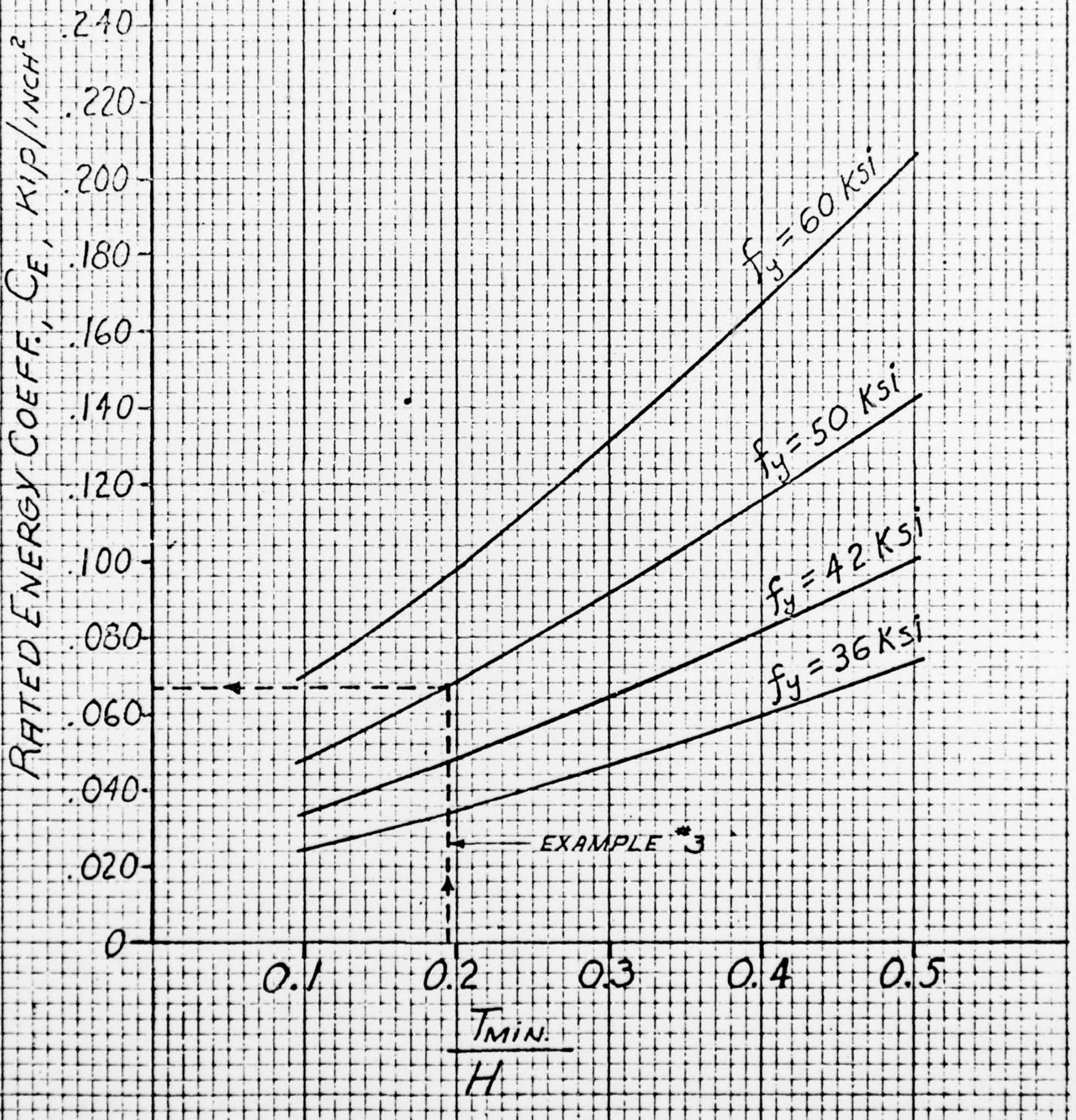
FIG. 2

59/60



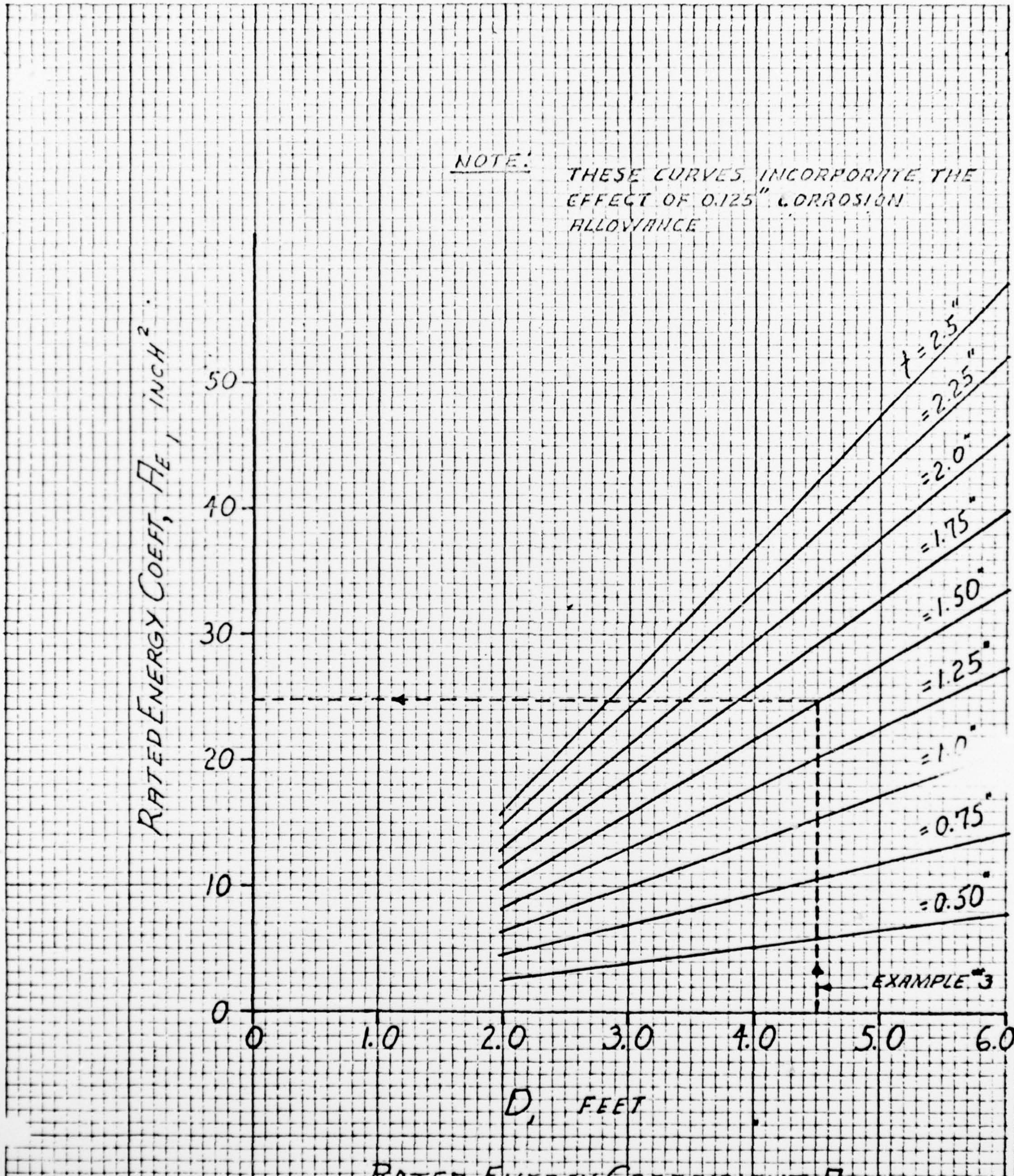
PILE CHARACTERISTIC LENGTHS

FIG. 3



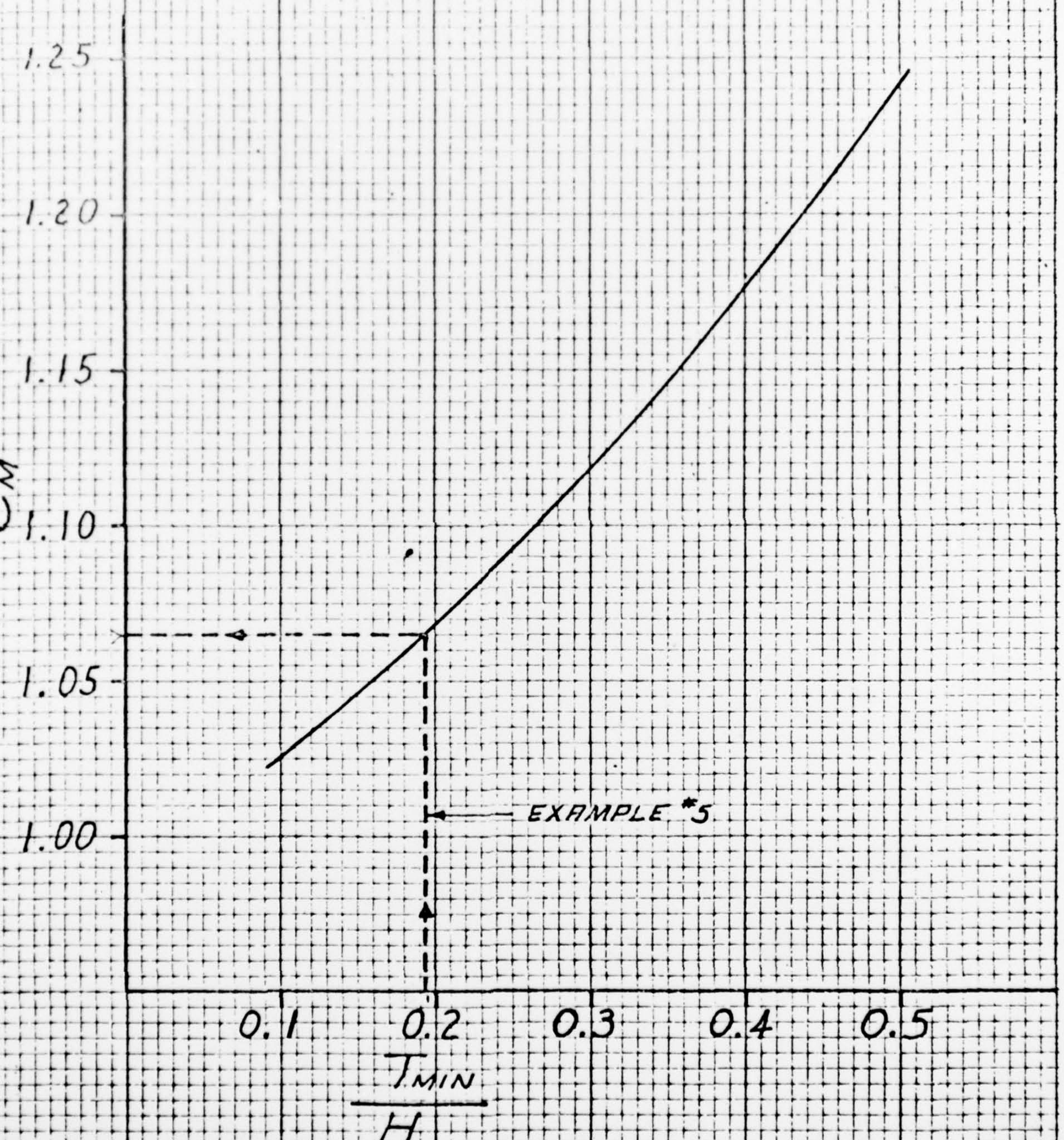
RATED ENERGY COEFFICIENT, C_E

FIG 4



RATED ENERGY COEFFICIENT, A_E

FIG 5



MAX. PILE MOMENT COEFFICIENT, C_M

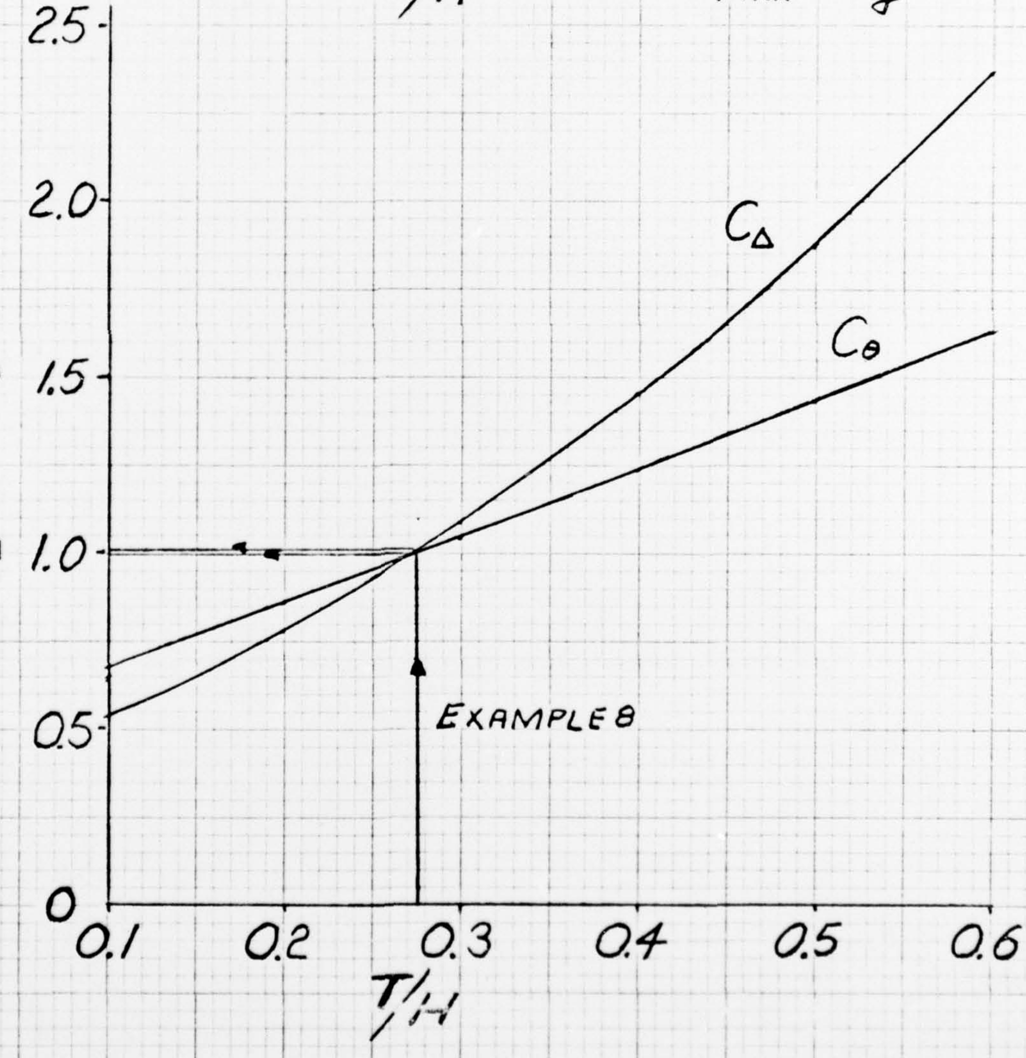
FIG 6

NOTES

1. PILE-TOP DEFL $= \Delta = C_\Delta \dfrac{H^2}{EI} M_{MAX}$

2. PILE-TOP ROTATION $= \theta = C_\theta \dfrac{H}{EI} M_{MAX}$

3. FOR MAX DEFL. & SLOPE, ENTER CHART WITH T_{MAX}/H AND USE $M_{MAX} = M_y$



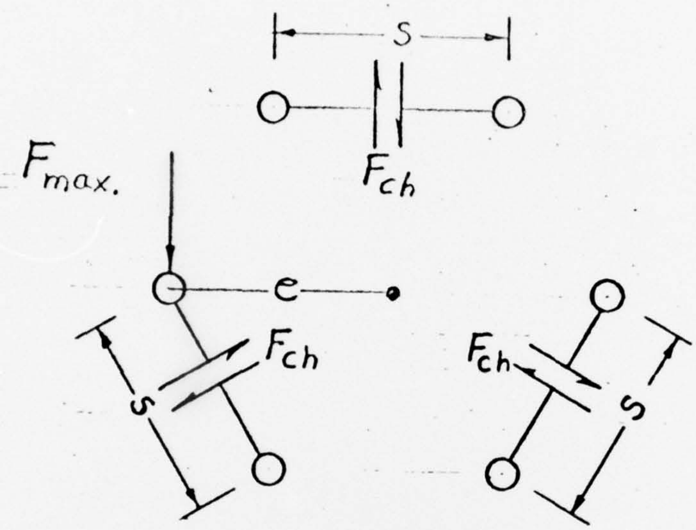
PILE DEFL. & SLOPE COEFFS, C_Δ & C_θ

REVISED 12/8/76

FIG 7

NOTE:

SYSTEM OF TENSION — COMPRESSION STRUTS NOT SHOWN ON ABOVE.



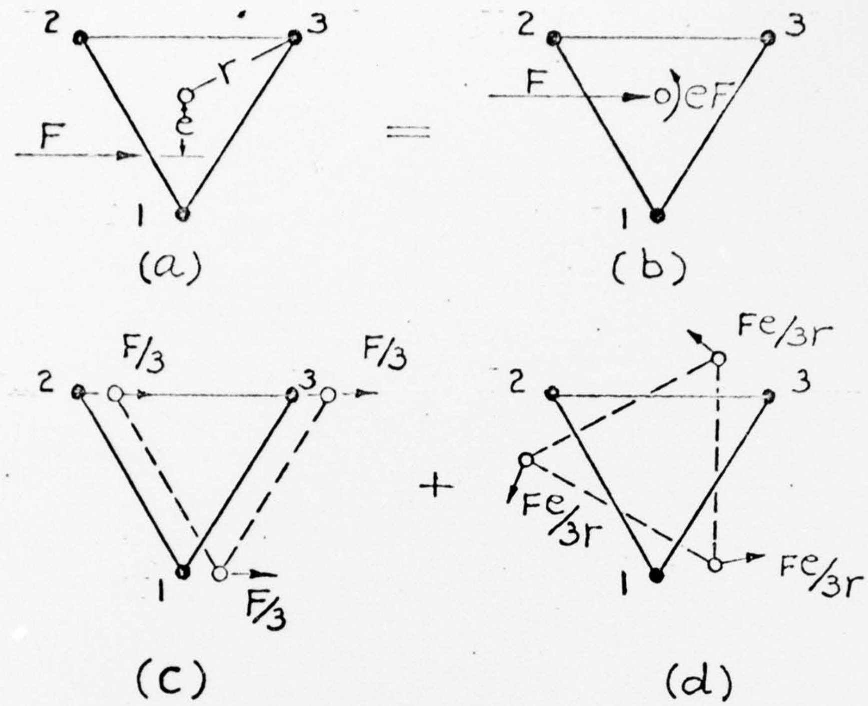
$$\text{AVERAGE } F_{ch} = \frac{2e}{S}\,\frac{F_{max}}{N_C} = \frac{F_{max}}{3}$$

$$\text{DESIGN } F_{ch} = \frac{3}{2}\,F_{ch}\text{ AVG.} = \frac{F_{max}}{2}$$

N_C = NUMBER OF CHAIN - CONNECTED PILES

CHAIN FORCES IN A 6-PILE CLUSTER WITH MOST SEVERE DOLPHIN LOAD ECCENTRICITY

FIG 8



FOR PILE 1, RESULTANT LOAD $= \frac{F}{3} + \frac{Fe}{3r}$

FOR PILES 2 & 3, " " $= \sqrt{\left(\frac{F}{3} - \frac{Fe}{3r}\cos 60°\right)^2 + \left(\frac{Fe}{3r}\sin 60°\right)^2}$

∴ PILE 1 IS THE MOST HEAVILY LOADED, AND CONTROLS THE DOLPHIN CAPACITY.

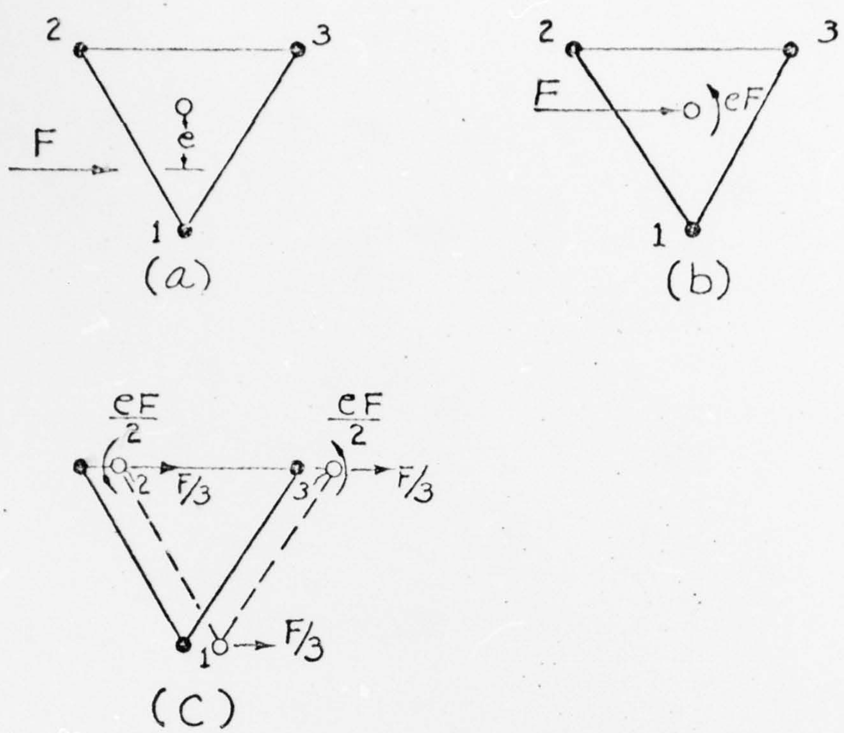
DOLPHIN CAPACITY, $F = \frac{3[\text{SINGLE PILE CAPACITY}]}{1 + e/r}$

NOTE:

DISCUSSION IN SECTION 6.2

PILE FORCES FOR 3-PILE DOLPHIN, IF DOLPHIN TORQUE IS RESISTED SOLELY BY PILE BENDING

FIG 9



RESULTANT LOAD = $\frac{F}{3}$ ON EACH PILE

PILE TORQUE $\frac{eF}{2}$ ON PILES 1 & 2

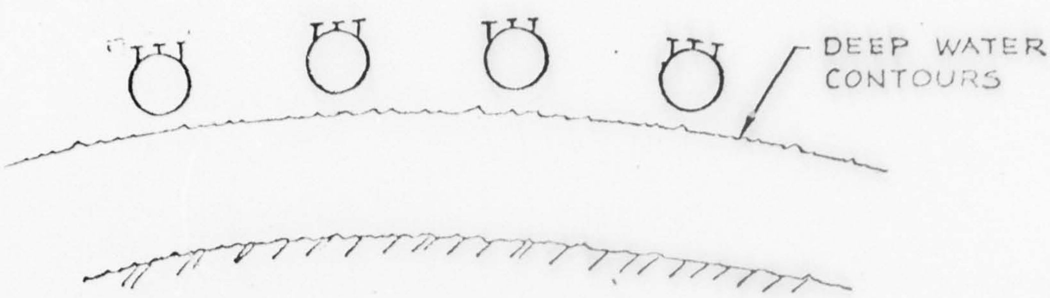
DOLPHIN CAPACITY, F = 3[SINGLE-PILE CAPACITY]

NOTE:

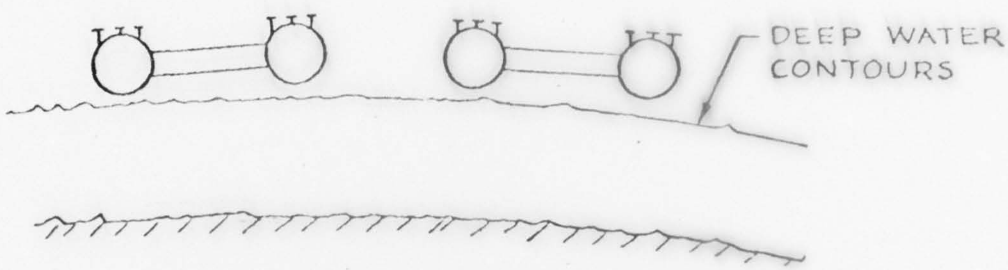
DISCUSSION IN SECTION 6.2

PILE FORCES FOR 3-PILE DOLPHIN, IF DOLPHIN TORQUE IS RESISTED SOLELY BY TORSION IN 2 INDIVIDUAL PILES

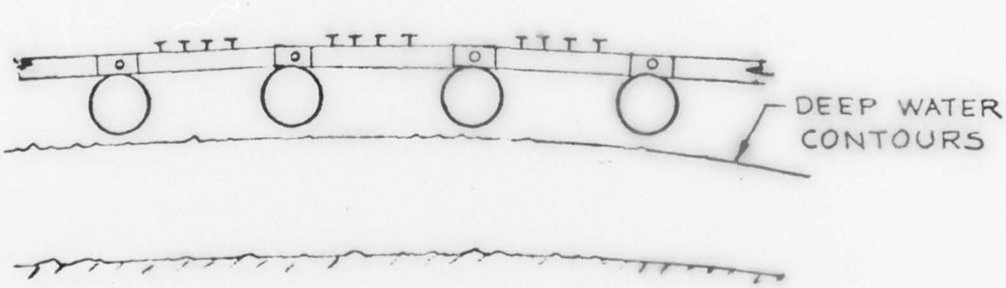
FIG 10



(a) SINGLE - PILE DOLPHIN SERIES



(b) MULTIPLE - PILE ALTERNATIVE



(C) MULTIPLE - PILE ALTERNATIVE

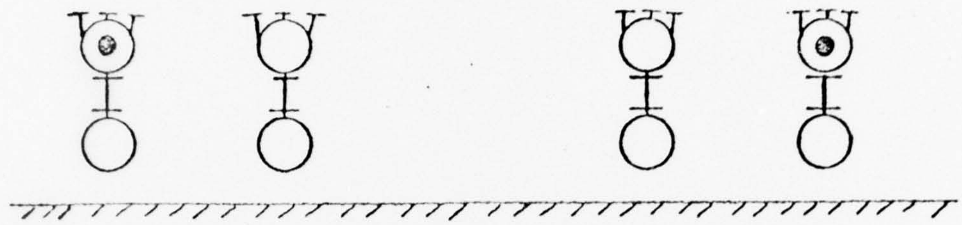
NOTE:

DISCUSSION IN SECTION 7.1.1

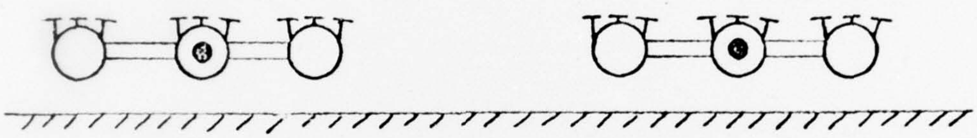
SINGLE - PILE AND MULTIPLE - PILE ALTERNATIVES FOR A SERIES OF DOLPHIN CONTACT POINTS

FIG 11

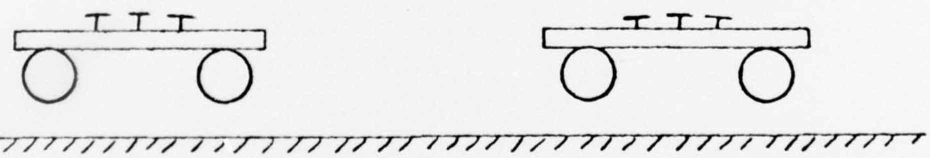
(a) ISOLATED SINGLE-PILE DOLPHINS



(b) 2-PILE DOLPHINS, WITH INTER-PILE STRUTS



(c) 3-PILE DOLPHINS, WITH INTER-PILE STRUT AND SHEAR CONNECTIONS



(d) 2-PILE DOLPHINS WITH FENDER BEAMS

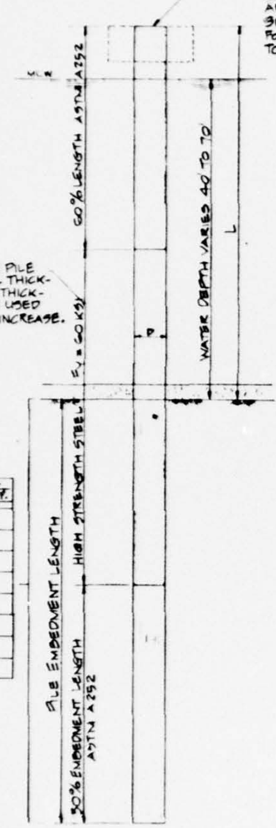
NOTE:

DISCUSSION IN SECTION 7.1.2
● = BOLLARD

ALTERNATIVE PILE ARRANGEMENTS FOR DOLPHIN CONTACT POINTS PARALLELING A PROTECTED FACILITY

FIG 12

I 80 B



DETAIL AT PILE TOP IS NOT PART OF THIS CONTRACT. IF APPLICATION REQUIRES LOADING AT ANY POINT ON THE PERIMETER, ARRANGEMENT WILL BE SIMILAR TO THAT SHOWN FOR SINGLE PILE DOLPHIN ON FIG. 14. FOR OTHER APPLICATIONS FENDER BRACKETING TO SUIT.

NOTE—
THIS LENGTH IS FOR PILE OF CONSTANT WALL THICKNESS. IF STEPPED THICKNESSES ARE TO BE USED THIS LENGTH WILL INCREASE.

PILE SIZE	EMBED-MENT DPT.
60 × .75	72'
60 × 1.00	76'
60 × 1.25	79'
60 × 1.50	82'
72 × .75	80'
72 × 1.00	84'
72 × 1.25	88'
72 × 1.50	91'

NOTES—
1. DOLPHIN RATINGS BASED ON 60 KSI YIELD GRADE STEEL IN REGION OF HIGH BENDING MOMENTS, 36 KSI YIELD GRADE IN OTHER REGIONS. OTHER STEELS OF EQUAL STRENGTH, DUCTILITY, WELDABILITY, AND RESISTANCE TO BRITTLE FRACTURE MAY BE SUBSTITUTED FOR THOSE INDICATED.
2. CHOICE OF WELDING RODS, PRE-HEAT AND POST WELD HEAT TREATMENT, IN THE EXECUTION OF GIRTH WELDS IN THE HIGH STRENGTH PILE STEEL, MUST BE CONSISTENT WITH THE STEEL SELECTED, AND WITH THE NECESSITY TO MINIMIZE THE POSSIBILITY OF BRITTLE FRACTURE.
3. GIRTH WELDS IN THE PILES ARE TO BE FULL PENETRATION AND FLUSH GROUND.

TABLE OF STATIC FORCE AND ELASTIC ENERGY CAPACITIES* (FOR LARGER DIA. SINGLE PILES)

NOMINAL WATER DEPTH	PILE SIZE (O.D. × THICKNESS)	RATED STATIC FORCE (KIPS)	RATED ELASTIC ENERGY (FOOT KIPS)	NOMINAL WATER DEPTH	PILE SIZE (O.D. × THICKNESS)	RATED STATIC FORCE (KIPS)	RATED ELASTIC ENERGY (FOOT KIPS)
40'	60" × .75"	74	85	60'	60" × .75"	55	72
40'	60" × 1.00"	100	97	60'	60" × 1.00"	75	107
40'	60" × 1.25"	125	151	60'	60" × 1.25"	95	144
40'	60" × 1.50"	153	167	60'	60" × 1.50"	116	185
40'	72" × .75"	106	84	60'	72" × .75"	79	92
40'	72" × 1.00"	145	130	60'	72" × 1.00"	108	141
40'	72" × 1.25"	180	175	60'	72" × 1.25"	136	189
40'	72" × 1.50"	212	222	60'	72" × 1.50"	165	258
50'	60" × .75"	65	68	70'	60" × .75"	49	76
50'	60" × 1.00"	86	100	70'	60" × 1.00"	67	113
50'	60" × 1.25"	109	137	70'	60" × 1.25"	84	151
50'	60" × 1.50"	134	175	70'	60" × 1.50"	101	192
50'	72" × .75"	91	88	70'	72" × .75"	71	97
50'	72" × 1.00"	124	155	70'	72" × 1.00"	97	147
50'	72" × 1.25"	155	162	70'	72" × 1.25"	122	198
50'	72" × 1.50"	189	229	70'	72" × 1.50"	146	249

* FOOTNOTES—
1. RATED STATIC FORCE—(APPLIED 10 FT ABOVE WATER SURFACE) IS 50% OF FORCE REQUIRED TO CAUSE PILE TO YIELD.
2. RATED ELASTIC ENERGY—(FOR CONCENTRATED LOAD AT NORMAL ELEVATION OF WATER SURFACE) BASED ON MAXIMUM STRESS IN PILE = .75 F_y. F_y = 60 KSI IN REGIONS OF MAXIMUM MOMENT.
3. RATED ENERGY CAPACITIES REFLECT DEFLECTIONS BASED ON CONSTANT WALL THICKNESS. WHERE INDICATED THICKNESS EXCEEDS .75 INCHES STEPPED REDUCTIONS IN THICKNESS IN ACCORDANCE WITH BENDING MOMENT ARE POSSIBLE. SUCH REDUCTIONS WOULD INCREASE PILE TOP DEFLECTIONS AND RATED ENERGY.

RATED CAPACITIES OF LARGE DIAMETER SINGLE PILE DOLPHINS
NAVAL FACILITIES ENGINEERING COMMAND
CONTRACT NO. N00025-71-C-0023

FIG. 13

REVISIONS
A
B
C

Drawn
Checked
Approved

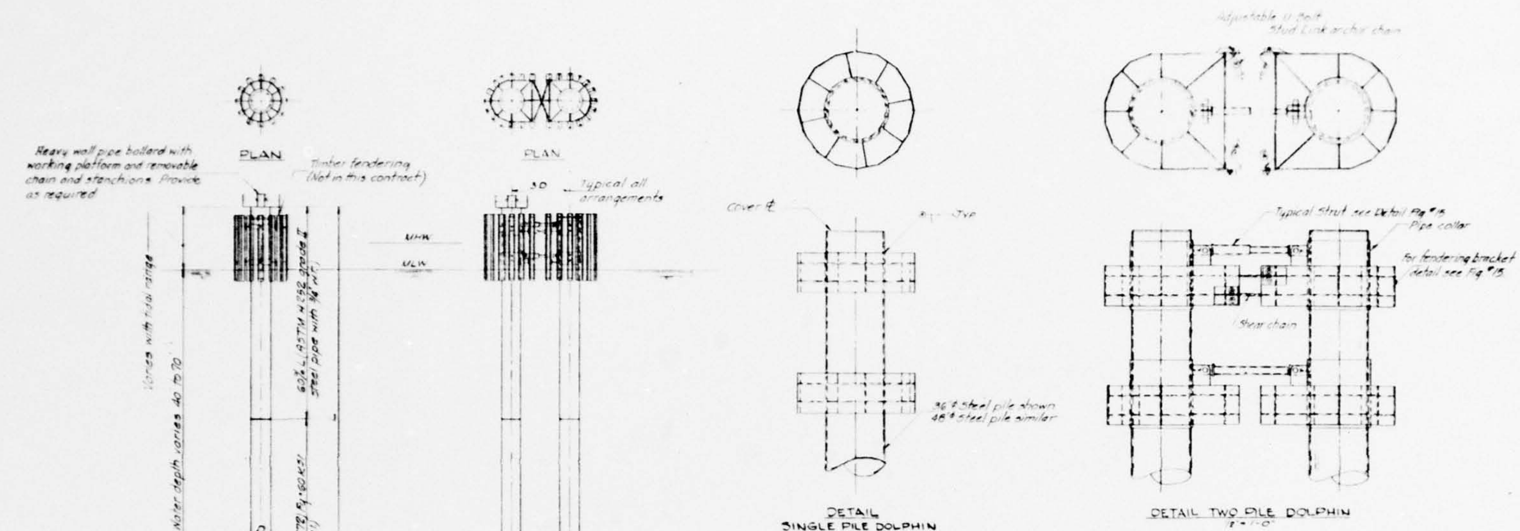


TABLE OF STATIC FORCE & ELASTIC ENERGY CAPACITIES				CHAIN SIZE
NOMINAL WATER DEPTH	NUMBER & SIZE OF PILES	RATED STATIC*1 FORCE (KIPS)	RATED ELASTIC*2 ENERGY (FOOT KIPS)	GRADE III STUD LINK ANCHOR CHAIN ABS OR LLOYDS CLASS
40	1-36" PP.75	27	30	—
40	2-36" PP.75	54	59	1 5/8"
40	2-48" PP.75	95	90	1 5/8"
40	4-36" PP.75	108	118	1 5/8"
40	5-36" PP.75	135	148	1 3/4"
40	4-48" PP.75	191	179	1 3/4"
50	1-36" PP.75	23	32	—
50	2-36" PP.75	46	64	1 5/8"
50	2-48" PP.75	81	96	1 5/8"
50	3-48" PP.75	122	144	1 3/4"
50	4-48" PP.75	163	192	1 3/4"
50	5-48" PP.75	204	240	1 3/4"
60	1-48" PP.75	36	52	—
60	2-48" PP.75	71	103	1 5/8"
60	3-48" PP.75	107	155	1 5/8"
60	4-48" PP.75	142	206	1 5/8"
60	5-48" PP.75	178	258	1 3/4"
60	6-48" PP.75	214	310	1 3/4"
70	1-48" PP.75	32	55	—
70	3-36" PP.75	53	113	1 5/8"
70	5-36" PP.75	88	188	1 5/8"
70	5-48" PP.75	158	276	1 3/4"
70	6-48" PP.75	189	331	1 3/4"
70	7-48" PP.75	221	386	1 3/4"

*FOOTNOTES:
1. RATED STATIC FORCE (APPLIED 10' ABOVE WATER SURFACE) IS 50% OF FORCE REQUIRED TO CAUSE PILE TO YIELD.
2. RATED ELASTIC ENERGY (FOR CONCENTRATED LOAD AT NOMINAL ELEVATION OF WATER SURFACE) BASED ON MAXIMUM STRESS IN PILE =.75 Fy, Fy=60 KSI IN REGIONS OF MAXIMUM MOMENT.

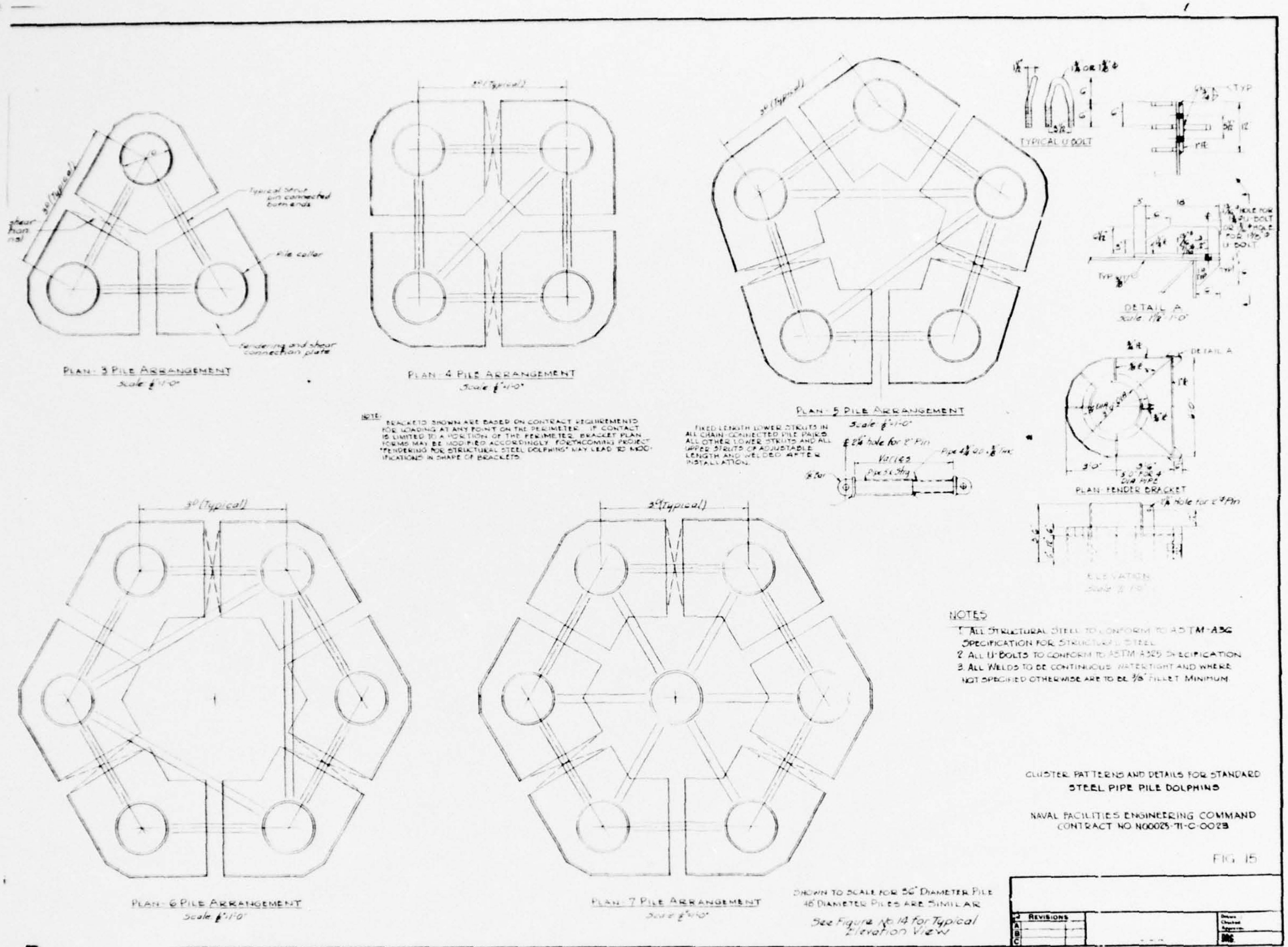
NOTES-
1. DOLPHIN RATINGS BASED ON 60KSI YIELD GRADE STEEL IN REGION OF HIGH BENDING MOMENT; 36KSI YIELD GRADE IN OTHER REGIONS. OTHER STEELS OF EQUAL STRENGTH, DUCTILITY, WELDABILITY AND RESISTANCE TO BRITTLE FRACTURE MAY BE SUBSTITUTED FOR THOSE INDICATED.
2. CHOICE OF WELDING RODS, PRE-HEAT AND POST-WELD HEAT TREATMENT IN THE EXECUTION OF GIRTH WELDS IN THE HIGH STRENGTH PILE STEEL MUST BE CONSISTENT WITH THE STEEL SELECTED AND WITH THE NECESSITY TO MINIMIZE THE POSSIBILITY OF BRITTLE FRACTURE.
3. GIRTH WELDS IN THE PILES ARE TO BE FULL PENETRATION AND FLUSH GROUND.

STANDARD STEEL PIPE PILE DOLPHINS

NAVAL FACILITIES ENGINEERING COMMAND
CONTRACT NO. N00025-71-C-0023

FIG. 14

REVISIONS	
A	
B	
C	



PLAN - 3 PILE ARRANGEMENT
Scale 1/8"=1'-0"
Typical strut pin connected both ends
Pile collar
Fendering and shear connection plate
PLAN - 4 PILE ARRANGEMENT
Scale 1/8"=1'-0"
PLAN - 5 PILE ARRANGEMENT
Scale 1/8"=1'-0"
NOTE: BRACKETS SHOWN ARE BASED ON CONTRACT REQUIREMENTS FOR LOADING AT ANY POINT ON THE PERIMETER. IF CONTACT IS LIMITED TO A PORTION OF THE PERIMETER, BRACKET PLAN FORMS MAY BE MODIFIED ACCORDINGLY. FORTHCOMING PROJECT "FENDERING FOR STRUCTURAL STEEL DOLPHINS" MAY LEAD TO MODIFICATIONS IN SHAPE OF BRACKETS.
FIXED LENGTH LOWER STRUTS IN ALL CHAIN-CONNECTED PILE PAIRS. ALL OTHER LOWER STRUTS AND ALL UPPER STRUTS OF ADJUSTABLE LENGTH AND WELDED AFTER INSTALLATION.
2¼ hole for 2" Pin
Varies
Pipe 5 x Strg
TYPICAL U BOLT
PILE
DETAIL A
PLAN - FENDER BRACKET
ELEVATION
PLAN - 6 PILE ARRANGEMENT
Scale 1/8"=1'-0"
PLAN - 7 PILE ARRANGEMENT
Scale 1/8"=1'-0"
SHOWN TO SCALE FOR 56" DIAMETER PILE
48" DIAMETER PILES ARE SIMILAR
See Figure No. 14 for Typical Elevation View
NOTES
1. ALL STRUCTURAL STEEL TO CONFORM TO ASTM-A36 SPECIFICATION FOR STRUCTURAL STEEL
2. ALL U-BOLTS TO CONFORM TO ASTM A325 SPECIFICATION
3. ALL WELDS TO BE CONTINUOUS, WATERTIGHT AND WHERE NOT SPECIFIED OTHERWISE ARE TO BE 3/8" FILLET MINIMUM
CLUSTER PATTERNS AND DETAILS FOR STANDARD STEEL PIPE PILE DOLPHINS
NAVAL FACILITIES ENGINEERING COMMAND
CONTRACT NO N00025-71-C-0023
REVISIONS

FIG. 15